FLOOD ROUTING THROUGH A RIVER WITH FLOOD PLAINS

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Dedicated

to

Parents

CERTIFICATE

It is certified that the work contained in the thesis entitled, "FLOOD ROUTING THROUGH A RIVER WITH FLOOD PLAINS", by Shri Sunil Kumar Choubey, has been carried out under my supervision and this work has not been submitted elsewhere for a degree.

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ABSTRACT

The object of the thesis is to study the role of flood plains on flood peak subsidence and time of occurrence of peak. A two-dimensional mathematical model for flood routing through a river with flood plains has been formulated. The model uses only continuity equation. By two-dimensional, it does not refer to the unsteady flow equations in two spatial dimensions, but rather to the physical situation in which river and flood plain cells form a two-dimensional network in the horizontal plane, but flow of water between two cells is one-dimensional. The results of the present model are compared with the results obtained by Mahapatra, K., (1990) for one-dimensional model with both continuity equation and equation of motion. Laws of exchanges of flow between two cells are governed either by river type link or weir type link depending on natural topography. River type link is based on Manning's equation, whereas weir type link follows the classical weir formulae. Since slope of channel is steep, bed slope has been used rather than water surface slope in Manning's formula.

The parameters governing the problem have been identified through a dimensional analysis. Values of $B_r = (B_s - B_m)/B_m = 4, 8$ and $N_r = n_2/n_1 = 1, 4$ have been used and the other parameters have been kept constant. The computer program has been developed in Fortran IV language and implemented on HP-9000 and micro-vax. From the computer results for all the sets of data it is found that with $n_r = 1.0$ model does not exhibit any flood peak subsidence as flood wave propagates down the river, whereas with $n_r = 4.0$ the model exhibits flood peak subsidence. After

comparing the results of the two dimensional model with the results of the one-dimensional model it is found that n_r has more effect in two-dimensional model on flood peak subsidence whereas B_r has more effect in one-dimensional model. The present study uses explicit method and some anomalies have occurred. It is recommended that the same problems should be studied using implicit method.

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Sunil Kumar Choubey

LIST OF FIGURES

LIST OF SYMBOLS

$A_{i,k}$ = Area of the flow cross section between cells i and k.

AS_i = Water surface area in horizontal plane of the i-th cell.

ΔAS_i = Increment of water surface area due to change in water level.

B_r = $(B_s - B_m)/B_m$.

B_m = Width of main channel.

B_s = Width of channel including flood plains.

Cdw = Coefficient of weir.

Cdo = Coefficient of orifice.

dt = Time increment.

dx = Distance increment.

dZ_i = Water level increment.

g = Acceleration due to gravity.

i,k = Subscripts indicating cells.

K = Conveyance of the cross section.

n_1 = Manning's n of main channel.

n_2 = Manning's n of flood plains.

n_r = n_2/n_1.

P_i = Rainfall intensity.

$Q_{i,k}$ = Flow exchange between cells i and k.

$R_{i,k}$ = The hydraulic radius of the flow cross section between cells i and k.

S = Water surface slope.

S_o = Bed slope.

t = Time under consideration.

t_{peak} = The time to peak stage of inflow stage hydrograph.

t_b = Time base of in flow hydrograph.

$\forall_i$ = Volume of water stored in cell i.

$\Delta\forall_i$ = Increment to volume of water stored in cell i.

X = Distance along the channel.

y_f = Depth below bottom of flood plain.

y_o = Steady state depth.

y = Water level in the river cells above river bed level at section under consideration.

y' = Water level in the flood plain cells above river bed level at that section.

Z_b = Bed level from datum.

Z_i = Water level in cell i.

Z_w = Elevation of water surface.

Zw_o = Steady state stage.

Zwo_p = peak stage of inflow hydrograph.

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CHAPTER I

INTRODUCTION

1.1 GENERAL

Mathematical modelling provides a tool by means of which one can:

(i) Study and gain an understanding of hydraulic flow phenomena.

(ii) Select and design sound engineering projects.

(iii) Predict extreme situations so as to be able to provide advance warning of their occurrence and importance, and

(iv) For proper management of flood plain area.

The prediction of exceptional natural events and their consequences implies the use of mathematical modelling because past observations of such events are so often non-existent. The capability of a properly constructed mathematical model to perform well depends on the fact that it uses physically correct, deterministic hydraulic equations whose validity is not limited to the known flow events used to adjust their empirical coefficients.

Flood Routing is basically determination of stage hydrograph at any section in the river if the hydrograph (stage or discharge) is given at an upstream location. There are two most important parameters in analysing the flood (i) Peak depth and (ii) time of

occurrence of peak depth. There are several methods available for predicting these parameters for any given inflow hydrograph and channel parameters. These models may be broadly classified under two headings:

(i) One-Dimensional Model and

(ii) Two-Dimensional Model

Truly straight channels in which the flow can be considered to be strictly one-dimensional are seldom observed in nature. It is more often the case that over bank flows go their own way, filling up the flood plain in a manner dictated by local topography, sometimes never returning to the main channel during the falling flood.

By two dimensional we do not refer to the unsteady flow equation in two spatial dimensions, but rather to the physical situation in which channel and storage cells form a two-dimensional network in the horizontal plane, but flow of water between two cells is one dimensional.

Mathematical modelling of unsteady flow in river is much more complicated than in artificial channels because of possible inundation of flood plain and consequently flow is not one-dimensional any more. The construction of the mathematical model of a flooded area sets up a series of problems concerning both the representation of the physical reality and the numerical methods. Many times it is not possible to predict the extent of inundation itself. Even with well defined flood plains the flow

pattern is not simple and interaction between the main river and the flood plain cells affect the flood wave propagation.

During the rising flood, the positive wave advancing along the river will have higher celerities and higher stages along its length. Thus there will be a transverse slope which will fill up the inundated plain with water. During the falling flood, the reverse situation will be observed: the main channel will try to drain first and the transverse slope will then cause water to flow from the inundated flood plain back to the main channel. But one dimensional modelling as described by the St. Venant equation written for non-prismatic channels implies that the transverse slope is nil. The lateral flow influences the celerity and consequently the time of occurrence of peaks (Cunge, 1975). The flood is also attenuated because of this. Therefore any accurate model should take into account these two-dimensional effects. If in the most situations purely one-dimensional models are used, it is because two-dimensional ones have historically been either too difficult or too expensive to construct and apply. It is fortunate that most often the overall results of these models do not seriously differ from reality, although they cannot always reproduce complicated details of flow which are all but one-dimensional. The present work is an endeavour to develop a two-dimensional mathematical model to study the role of flood plains on the flood wave propagation in a river with flood plains and also to compare it with one dimensional model.

1.2 REVIEW OF LITERATURE

Mathematical modelling of unsteady flow for ideal condition which uses the complete equations is very common and widely covered in several text books (Cunge, et.al., 1980, Chaudhary, 1987). However, technique finds its origin in the 19th Century work of de St. Venant and Boussinesq who formulated the unsteady flow equations, and in the work of Massau who in 1889 published some early attempts to solve those equations. Most of the methods are applicable to only ideal situations like simple channel. Very few models have been developed to meet practical situations. Important theoretical concepts were established in first half of this century, but the first engineering applications of this principle to the natural river conditions awaited the development of digital computers. Isaacson, E., Stoker, J., and Troesch, B.A., (1954) constructed and ran a mathematical model of portion of the Ohio and Mississippi rivers by means of finite difference method on the digital computer, UNIVAC. Special attention was given to problems resulting from junction of the Ohio and Mississippi rivers. Following that pioneering effort the use of mathematical modelling in natural river began to accelerate. Cunge, J.A., (1966) presented a paper discussing the difficulties encountered in the development of mathematical models for the reach of Oraison canal. He compared the mathematical model with scale model and comparative results were presented. He concluded that mathematical model results are quite comparable with those

given by a distorted scale model and use of mathematical model in preference to distorted scale models is justified.

Later on Cunge, J.A., Lorgere, H., and Preissmann, A., (1966) constructed the UNESCO mathematical model for the control of floods of the river Mekong. Zanobetti, D., Lorgere, H., Preissmann, A., and Cunge, J.A., (1968) were involved in the development and application of SOGREAH'S CARIMA system for the simulation of unsteady flow in Mekong river and Mekong flood plains. In 1970, Zanobetti, D., Lorgere, Henri, Preissmann, Alexandre and Cunge, J.A., constructed and used the model for lower basin of the Mekong river. The model consisted of a system of ordinary first order differential equations each representing water level as a function of time in one of the some 300 meshes in to which whole area was divided. Exchange relations between cells were based either on Saint Venant's dynamic equations or on the Weir type exchange laws, inertia terms being ignored in both cases due to the very small velocity of flood propagation. Cunge, J.A., (1971) presented a paper in proceedings, 14th Congress, IAHR, Paris discussing the representation of flood plains in the mathematical models in rivers. Grijsen and Vreugdenhil, (1976) developed a mathematical model of the Rharb plain in Morocco. (Basically mathematical formulation is as in one dimensional case). In two- dimensional model they considered inter connection of one cell with its neighbouring cells. In the case of the Oused Sebou and the Rharb plain, the network approach was used and full dynamic equations including inertial effects were used. But they

realised that if there is gradual variation of discharge or stage, then inertia does not play an important part. Cunge also concluded similar idea about neglecting inertial effect.

Tingsanchali and Ackerman (1976) derived equations for compound channel flow which include both dynamic and storage effects of the over bank flow. They applied these equations to study the floods in a natural river and concluded that higher peak stages and lower peak discharges are obtained when only storage effects of flood plains is considered. Later Tingsanchali and Lal (1988) using the same equations obtained semiemperical exponential equations to describe the subsidence of flood peak discharge in channel during over bank flow periods.

Finally, Mahapatra, K., (1990) routed the flood through compound channel using inertial effect and concluded that one can neglect inertial effect for natural river and flood plains. He recommended for development of two-dimensional mathematical model of compound channel ignoring the inertial effects.

1.3 SCOPE OF PRESENT STUDY

1. To present a two-dimensional mathematical model of a river with flood plains. Inertial effect has been ignored from dynamic equation. An explicit finite difference method is used to solve governing equation numerically.
2. To study the role of channel parameters on the flood wave propagation.

3. To do comparative study with one-dimensional model with inertial effect developed by Mahapatra, K., (1990).

1.4 CLOSURE

The following chapters present the development of two-dimensional mathematical model of river with flood plains. Chapter II presents governing equations and method of solution. Assumptions made are also given in Chapter II. Chapter III explains the application of model to a river with flood plains results and discussion. Chapter IV includes conclusion and recommendations for future work. The computer programme has been included in Appendix A.

CHAPTER II

GOVERNING EQUATIONS AND METHOD OF SOLUTION

2.1 INTRODUCTION

In this chapter the governing equations for unsteady flow without inertial effect have been presented. In addition to this method of numerical solution to these equations has been given.

The arrangement in this chapter is as follows. In Section 2.2 assumptions made in the derivation of the equations are discussed. In Section 2.3 continuity equation for a cell and laws of discharge between the cells are presented. In Section 2.4 numerical solution of governing equations are presented and in Section 2.5 accuracy and stability are discussed.

2.2 ASSUMPTIONS

The assumptions made for deriving the governing equations are as follows:

(i) There are gradual variations in flow characteristics, so that one can neglect inertial effect.

(ii) The volume $\forall_i$ of water stored in cell i is directly related to the level Z_i in this cell i.e.,

$$\forall_i = \forall\,(Z_i)$$

(iii) The discharge $Q^n_{i,k}$ between two adjacent cells i and k at the given time $t_n = n \Delta t$ is a function of the levels Z^n_i and Z^n_k only, i.e.

$$Q^n_{i,k} = Q (Z^n_i, Z^n_k)$$

(iv) The discharge between two cells $Q_{i,k}$ does not vary during the period of time Δt.

(v) The slope, θ of the channel bed slope is small so that $\sin \theta \simeq \tan \theta \simeq \theta$ and $\cos \theta \simeq 1$.

(vi) The resistance coefficient for unsteady flow is same as for steady and uniform flow.

(vii) The velocity distribution in a vertical plane is uniform.

2.3 DERIVATION OF EQUATIONS

In this section continuity equation for a cell and law of discharge between cells are derived. The derivation closely follows the derivation given by Cunge J.A. (Chapter 17, K. Mahmood and V. Yevjevich).

2.3.1 Continuity Equation for a Cell

Let us consider a cell as shown in Fig. 2.1.

Suppose that to every cell i corresponds a characteristic water level Z_i assumed at the center of the cell. It is also assumed that the water surface is horizontal within the cell boundaries and its elevation above the datum is Z_i.

A constant time step Δt is assumed. Then, for any given time

A constant time step Δt is assumed. Then, for any given time $t_n = n\Delta t$ the water in cell i is $Z_i(tn)$ and the corresponding water surface area (the surface ABCD) is equal to $AS_i \ (t_n)$ the water surface area in the horizontal plane of the cell.

At the next time step $t_{n+1} = t_n + \Delta t$ the water level is Z_i (t_{n+1}) and water surface area is $As_i(t_{n+1}) = AS_i(t_n) + \Delta AS_i$. The increment of water level may be due to runoff $P_i(t)$ resulting from rainfall $P_i(t)$ on the cell surface during the time Δt and to discharge $Q_{i,k}(Z_i,Z_k)$ from the cells K adjacent to the cell i. The increase in water volume stored in the cell i during the time Δt from discharge conditions is

$$\Delta \forall_i = \int_{t_n}^{t_n+\Delta t} P_i(t)\ dt + \sum_k \int_{t_n}^{t_n+\Delta t} Q_{i,k}\ (Z_i,\ Z_k)\ dt \qquad (2.1)$$

where, the suffix k represents numbers of all cells adjacent to cell i. From geometric consideration, the above relation can be written as below

$$\Delta \forall_i = \int_{Z_i(t_n)}^{Z_i(t_n+\Delta t)} AS_i\ (Z_i)\ dZ_i \qquad (2.2)$$

Suppose that the surface area AS_i does not change between the levels Z_i and $Z_i + \Delta Z_i$ i.e., $\left(\frac{\Delta AS_i}{AS_i} \ll 1 \right)$.

From Fig. 2.1,

$$\Delta \forall_i = V_{ABCDA'B'C'D'} = V_{ABCDA''B''C''D''} = AS_i(Z_i)\ \Delta Z_i$$

This relationship replaces integral sign in equation (2.2) and dZ_i is replaced by ΔZ_i.

Equating equations (2.1) and (2.2)

$$AS_i \; (Z_i) \; \Delta Z_i = P_i \; (\tau).\Delta t + \Delta t \sum_k Q_{i,k} \; \{Z_i(\tau), \; Z_k(\tau)\} \qquad (2.3)$$

where,

$n.\Delta t \leq \tau \leq (n+1) \; \Delta t$

If $\Delta Z \rightarrow 0$, and $\Delta t \rightarrow 0$.

Equation (2.3) can be written under the differential form

$$AS_i \frac{dZ_i}{dt} = P_i(t) + \sum_k Q_{i,k} \; (Z_i, \; Z_k) \qquad (2.4)$$

This is the continuity equation for cell i.

Similar equation can be written for each cell. Thus for N cells there would be N ordinary differential equations for N unknown functions Z_i of the independent variable t.

2.3.2 Laws of Discharge between the Cells

Generally two types of connections between cells are assumed. These are:

(i) River type link

(ii) Weir type link

River type links represent the exchanges between two cells when there is no local obstacle to the flow and a mean resistance co-efficient for a given cross section of the flow can be used.

Weir type links represent exchanges between cells where roads or dikes form a local obstacle that can be represented by singular head loss between two cells.

2.3.2.1 River type link:

In the case of river type link Manning's formula is used,

$$Q_{i,k} = \frac{1}{n} \cdot A_{i,k} \cdot R_{i,k}^{2/3} \cdot S^{1/2} \qquad (2.5)$$

where,

n = Manning's roughness coefficient

$A_{i,k}$ = the area of the flow cross section between cells i and k

$R_{i,k}$ = the hydraulic radius of the flow cross section between cells i and k, and

S = water surface slope.

In equation (2.5) parameters n, $A_{i,k}$ and $R_{i,k}$ are function of water level $\bar{Z}_{i,k}$ in the flow section between cell i and k. We can write

$$\frac{1}{n} \cdot A_{i,k} \cdot R_{i,k}^{2/3} = K(\bar{Z}_{i,k})$$

where,

$$\bar{Z}_{i,k} = \alpha Z_i + (1-\alpha) Z_k$$

$\bar{Z}_{i,k}$ is weighted level between levels in two cells i and k and $K(\bar{Z}_{i,k})$ is called as conveyance.

α is called weighting coefficient and it is constant for a given pair of cells. For equal sizes of cells, for given pair α is equal to 0.5. For bigger size of cells weighting coefficient

more than 0.5 is provided for that cell.

The water surface slope S in equation (2.5) is given as

$$S = \frac{Z_k - Z_i}{\Delta x}$$

where,

Δx is the fixed distance between the centers of cell i and k.

Let us define a function

$$\phi(\bar{Z}_{i,k}) = \frac{k(\bar{Z}_{i,k})}{\sqrt{\Delta x}} = \frac{AR^{2/3}}{n.\sqrt{\Delta x}}$$

where,

k is conveyance of the cross section.

The discharge formula can be written as

$$Q_{i,k} = \text{sign}\ (Z_k - Z_i)\ \phi\ .\ \sqrt{(Z_k - Z_i)}$$

Sign will depend upon the convention adopted with regard to the direction of flow. The function $\phi(Z_k, Z_i)$ must be established first based on the channel parameters.

2.3.2.2 Weir type link

In this case the classical discharge formulae for weirs are used.

There are two types of flow, namely submerged flow and free flow depending on following criterion:

Considering Fig. 2.2(a)

if $w < \frac{2}{3}(w+h)$, the flow is free flow and discharge is given by

$$Q_{i,k} = \frac{2}{3}\ .\ \sqrt{2g}\ .\ Cdw\ .\ dx\ .\ h^{3/2} \qquad (2.6)$$

if $w > \frac{2}{3}(w+h)$, the flow is submerged flow and discharge is given by

$$Q_{i,k} = \frac{2}{3}.\sqrt{2g}.Cdw \;.\; dx \;.\; h^{3/2} + \sqrt{2g}.Cdo \;.\; dx \;.\; w\sqrt{h} \qquad (2.7)$$

where,

dx = length of Weir (each cell)

h,w = as shows in Fig. 2.2(a)

Cdw and Cdo are coefficient of Weir and coefficient of orifice and given by Rouse as follows

$Cdw = 0.611 + 0.075\ h/w$

$Cdo = 0.611 - 0.175\ (w/h+w)$

2.4 NUMERICAL SOLUTION OF GOVERNING EQUATIONS

Introduction

With the advent of digital computers, more emphasis is being given on numerical solution methods. However, there have been some analog solutions. While using the numerical scheme, one should bear in mind the convergence, stability, efficiency and accuracy of the scheme before selecting it for a particular application. The governing equations have become very simple after dropping momentum equations. It has been left only with continuity equations and they are a set of ordinary differential equations. So, there is no need to use any special numerical technique. The set of differential equations can be solved by finite difference method either by explicit method or implicit

method depending on the assumption made in variation of $Q_{i,k}$ during the period of time Δt.

2.4.1 Numerical formulation

There are two ways of writing last term of equation (2.3) based on different assumption e.g. (i) Explicit method (ii) Implicit method.

2.4.1.1 Explicit method

It may be assumed that the discharge $Q_{i,k}$ does not vary during the period of the time Δt, i.e.

$$\tau = n.\Delta t$$

and

$$Q_{i,k}\{Z_i(\tau),\ Z_k(\tau)\} = Q_{i,k}\{Z_i(n.\Delta t),\ Z_k(n.\Delta t)\} = Q^n_{i,k}$$

In this case equation (2.3) becomes

$$\Delta Z_i\ .\ AS_i(Z^n_i) = P^n_i\ .\ \Delta t + \Delta t \sum_k Q^n_{i,k} \qquad (2.8)$$

It means, if the levels in the neighbouring cells are known at time tn = n.Δt. It gives an explicit solution for ΔZ_i, because all terms on right hand side of equation are known, then the resulting equation can be solved for each cell at a time.

Consequently, the water level at next time step $(n+1)\Delta t$ will be

$$Z^{n+1}_i = Z^n_i + \Delta Z_i$$

2.4.1.2 Implicit method

It may be assumed that the discharge $Q_{i,k}(Z_i(\tau),Z_k(\tau))$ is an intermediate discharge between $Q^n_{i,k}$ and $Q^{n+1}_{i,k}$. In other words,

$$n\Delta t \leq \tau \leq (n+1)\ \Delta t$$

$$Q_{i,k}(Z_i(\tau),Z_k(\tau)) = \theta Q^{n+1}_{i,k} + (1-\theta)\ Q^n_{i,k} \tag{2.9}$$

The equation (2.3) becomes

$$\Delta Z_i.AS_i = P_i\ \Delta t + \Delta t\ \left\{\theta \sum_k Q^{n+1}_{i,k} + (1-\theta) \sum_k Q^n_{i,k}\right\} \tag{2.10}$$

where, $0 \leq \theta \leq 1$.

In this case the water stages at next time step Z_i^{n+1} cannot be determined directly from equation (2.10) because its right hand side contains the value of unknown functions at time $(n+1)\Delta t$.

Since, the inter cell exchange laws (see equation river link, Weir link) are nonlinear and their substitution into equation (2.10) would further complicate the problem. For each cell there would be one nonlinear algebraic equation like (2.10). Since the model will have several hundred cells, consequently a large number of nonlinear equations will be obtained. It would not be a easy job to solve a large number of algebraic equations. In order to make the method of solution easier, the difference equations are linearized using the Taylor's series expansion and neglecting higher order terms with an assumption that the water level variations ΔZ_i are small during the time interval Δt.

$$Q^{n+1}_{i,k} = Q^n_{i,k} + \frac{\partial Q^n_{i,k}}{\partial Z_i}\ \Delta Z_i + \frac{\partial Q^n_{i,k}}{\partial Z_k}\ \Delta Z_k \tag{2.11}$$

Thus, a system of linear equations for the ΔZ_i is obtained.

After substituting the value of $Q_{i,k}^{n+1}$ in to equation (2.10)

$$AS_i^n \frac{\Delta Z_i}{\Delta t} = P_i + \sum_k Q_{i,k}^n + \theta \left\{ \sum_k \frac{\partial Q_{i,k}^n}{\partial Z_i} \Delta Z_i + \sum \frac{\partial Q_{i,k}^n}{\partial Z_k} \Delta Z_k \right\} \qquad (2.12)$$

Since the above equation contains two unknowns ΔZ_i and ΔZ_k, one cannot get ΔZ_i directly from it. Similar equation can be written for each cell and system of linear algebraic equations is obtained. The system of finite difference used in Eq. (2.12) is the implicit type depending on the value of weighing coefficient θ to express the derivatives. This is very important as far as the numerical solution is concerned.

By putting θ = 1 equation (2.12) becomes

$$AS_i^n \frac{\Delta Z_i}{\Delta t} = P_i + \sum_k Q_{i,k}^n + \sum_k \frac{\partial Q_{i,k}^n}{\partial Z_i} \Delta Z_i + \sum_k \frac{\partial Q_{i,k}^n}{\partial Z_k} \Delta Z_k \qquad (2.13)$$

To find the values of ΔZ_i and ΔZ_k the algebraic equations for the entire system of cells have to be solved simultaneously. Implicit finite difference schemes usually do not have to satisfy time constraints and therefore, longer computation time steps can be used. They involve matrix inversion and require large number of calculations, somewhat masking the advantage gained by using longer time steps.

2.4.2 Numerical Solution of Difference Equations

As seen in the Section 2.4.1, the continuity equation (2.3) can be written in two ways, either in explicit way or in implicit

way depending on assumption made for $Q_{i,k}$. For solving these two equations there are correspondingly two methods namely (i) Explicit method, (ii) Implicit method.

2.4.2.1 Explicit method

Rewriting the explicit type equation (2.8)

$$\Delta Z_i \; AS_i \; (Z_i^n) = P_i^n \; \Delta t + \Delta t \sum_k Q_{i,k}^n$$

$$\frac{dZ_i}{dt} AS_i = P_i^n + \sum_k Q_{i,k}^n$$

As it is clear from the above equation, ΔZ_i can be obtained without any need of other equations. Its solution is nothing but a solution of ordinary differential equation. So any numerical method for solving O.D.E. is sufficient. Thus explicit method simplifies the problem drastically but it has got a great demerit relating to stability. The price paid for this simplification is the limited choice of time step Δt. So, a considerable amount of computer time would be necessary. Thus for long period analysis implicit method is preferred in order to obtain a free choice of time step Δt.

2.4.2.2 Implicit method

Rewriting the implicit type equation (2.13)

$$AS_i \frac{\Delta Z_i}{\Delta t} = P_i + \sum_k Q_{i,k}^n + \sum_k \frac{\partial Q_{i,k}^n}{\partial Z_i} \Delta Z_i + \sum_k \frac{\partial Q_{i,k}^n}{\partial Z_k} \Delta Z_k$$

$$\Delta Z_i \left\{ - \frac{AS_i}{\Delta t} + \sum \frac{\partial Q_{i,k}^n}{\partial Z_i} \right\} + \sum_k \frac{\partial Q_{i,k}^n}{\partial Z_k} \Delta Z_k + L_i = 0 \qquad (2.14)$$

where,

$$L_i = P_i + \sum_k Q^n_{i.k}$$

The cells are arranged in certain number of groups as shown in Fig. 2.3 in such a way that each group is related only to the group directly above and below it.

Equation similar to (2.14) can be written for the central group j, the preceding group j-1 and the following group i+1. For details one may go through Chapter 17 of "Unsteady Flow in Channels", Vol. II,edited by K. Mahmood and V. Yevjevich.

CHAPTER III

TWO-DIMENSIONAL MATHEMATICAL MODEL FOR FLOOD ROUTING THROUGH A RIVER WITH FLOOD PLAINS

3.1 GENERAL

Unsteady flow in a river with flood plains is usually two-dimensional if the flow depth in the flood plain is shallow. By two dimensionality, we refer to the physical situation in which cells form a two-dimensional network and the exchanges of water between cells are purely one dimensional. For exact analysis, two dimensional modelling is required. Due to very limited time a channel with well defined flood plain has been chosen for study.

The aim of present work is to study the effect of variation of parameters of flood plain characterizing the geometry and flow condition during over bank flow periods. In addition to this, the results of this model are compared with the one dimensional model developed by Mahapatra, K. (1990) in which he has taken inertia terms into consideration. Inertia terms have been neglected assuming that variation in flow characteristics is gradual which is more likely to be practical situation.

3.2 CHANNEL GEOMETRY

As shown in Fig. 3.1, a symmetric prismatic channel with flood plains has been chosen for the study, primarily with a view to compare the results of the two-dimensional model with the results of the one-dimensional model studied by Mahapatra. The depth below the flood plain bottom is 5 m. There is no transverse bed slope in the main channel as well as on flood plain. The longitudinal bed slope for main channel and flood plain are same. The total length of channel reach is 10 km but only first 8 km has been studied to prevent any back water effect or down stream boundary effect.

In Fig. 3.1,

B_m = width of main channel

B_s = width of channel including flood plains

y_f = depth below flood plains

y = depth of flow

z_w = elevation of water surface

z_b = bed level from datum.

3.3 INITIAL CONDITION

The depth of flow at the center of each cell is defined at a given time (t = 0). Initially steady uniform flow is assumed with water level slightly more than the level of bottom of flood plains. The exchanges of flow have been computed based on river type link or weir type link depending on physical topography. River type link is governed by Manning's equation. The weir type

link is governed by Weir formula given by Rouse. The channel bottom slope is assumed to be constant along whole reach and throughout the duration of interest.

3.4 BOUNDARY CONDITION

3.4.1 Up Stream

For the present study a symmetric triangular shape of stage hydrograph of 1 hour has been considered at the upstream boundary cells. The shape of hydrograph is shown in Fig. 3.2,

where,

Z_{wo} = steady state stage

Z_{wop} = peak stage

t_{peak} = the time to peak stage

t = time under consideration

t_b = time base of hydrograph

The water level in all cells at upstream boundary is same at any time. It means there is no transverse water surface slope in upstream cells. The new water surface level is calculated directly from stage hydrograph for new time.

3.4.2 Down Stream

The new water level at down stream cell is computed by assuming that the change in water level in down stream cell is same as that of just preceding cell.

3.5 MODEL BUILDING

The division of an area like river with flood plains into individual cells, some of which are portions of main river and

other of the flood plain must be done carefully. The breakdown of given area into cells is based on natural topography. Since in present case, there are well defined flood plains and main river, division of an area into cells is not a difficult task. The cells have been divided into four categories. They are main river cells, next to the river flood plain cells, Intermediate flood plain cells, and Side boundary cells. After dividing the whole area into cells the picture looks like Fig. 3.3.

As shown in Fig. 3.3 each cell is linked with neighbouring cells with a link either river type or weir type depending on geographical topography. Since there is abrupt change in bed level between the river cells and just next to river cell, so they are connected with Weir type link because there would be singular head loss. Remaining links are river type links. River cells carry high flow compared to flood plains cells. The cell centers are their geometrical centers. However it is not necessary condition and it should be defined in such a way that the directions of flow are correctly allowed for any time during flood. The sizes of the flood plain cells and river cells are same and rectangle in shape of dimensions 500 m x 50 m. The bed level and water level have been assigned at the center of cell and assumed horizontal throughout the cell. The water surface area of cell is not a function of water level but it is constant. The manning's n is not a function of stage but constant. The side wall has same value of manning's n as its neighbouring bed surface. In river type link equation bed slope has been used for

flow in the longitudinal direction whereas water surface slope has been used for flow in the transverse direction.

3.6 NEW DEPTH COMPUTATION

After neglecting the rainfall terms in equation (2.8), it becomes

$$\frac{\Delta Z_i}{\Delta t} \cdot AS_i = \sum_k Q^n_{i,k} \tag{3.1}$$

Above equation is an algebraic equation and the value of ΔZ_i is found explicitly from given relation.

For any interior cell, all the discharges connected to that cell with neighbouring cells are found based on water level of cells at known time step and the type of link. These discharges are algebraically added which gives the value of term on right hand side of equation (3.1). Since AS_i and Δt are known, the value of ΔZ_i is found very easily with the following relation

$$\Delta Z_i = \sum_k Q^n_{i,k} \frac{\Delta t}{AS_i}$$

The same procedure is repeated with all the cells. If all the cells are exhausted, the new values are reset and time is increased by Δt. Same procedure is repeated for all the cells till total time of computation.

3.7 STABILITY CONDITION

Since explicit method has been used for solution of the difference equation, it is very much susceptible to unstability. The finite difference scheme is said to be stable if small

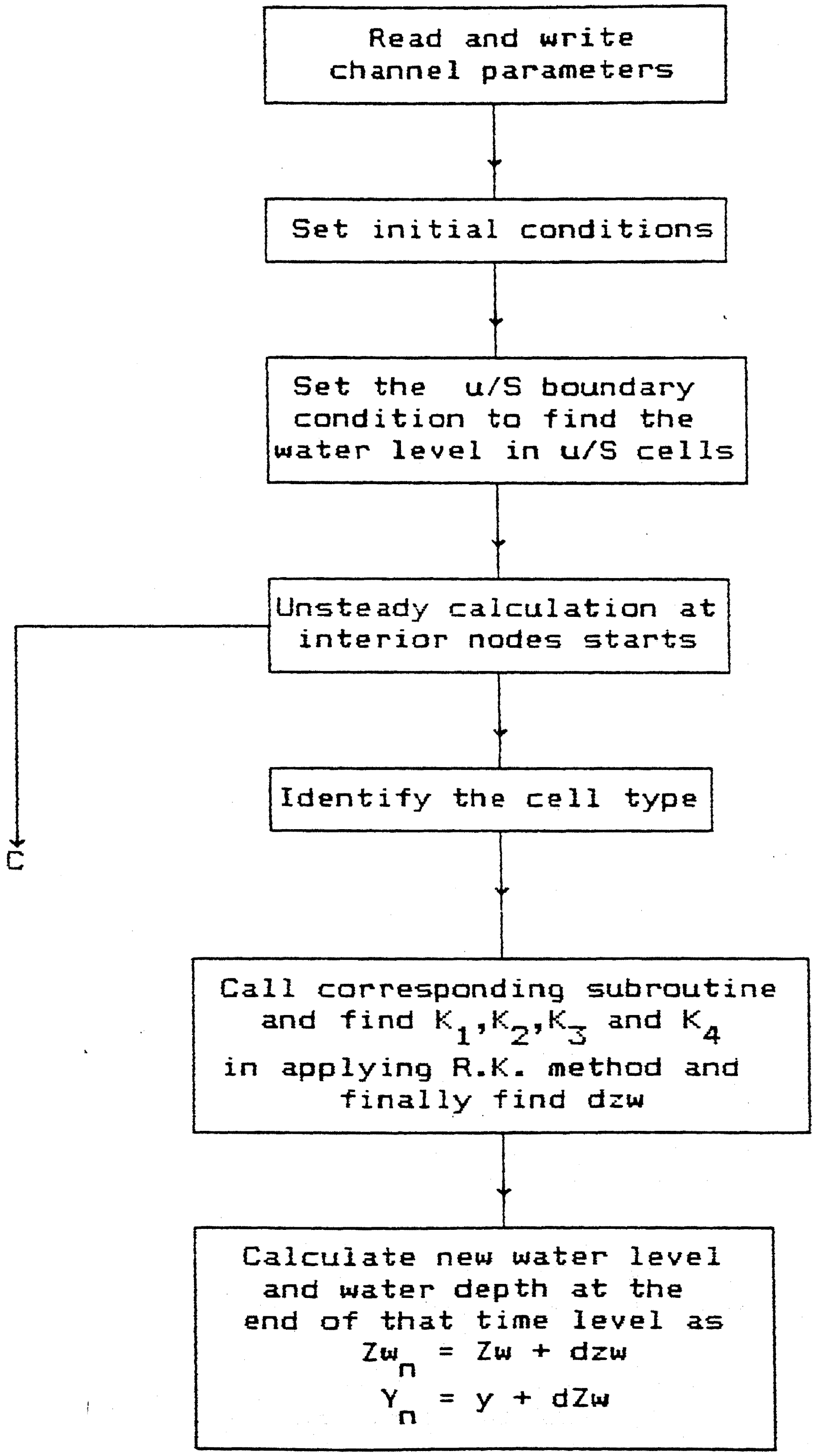
numerical error introduced at the beginning are not magnified during successive application of the difference equations and the errors at beginning do not grow so large that valid-part of the solution is obscured. So in explicit method of solution one will have very limited choice of selection of Δt and at the same time one should remain very much cautious while selecting the time interval Δt.

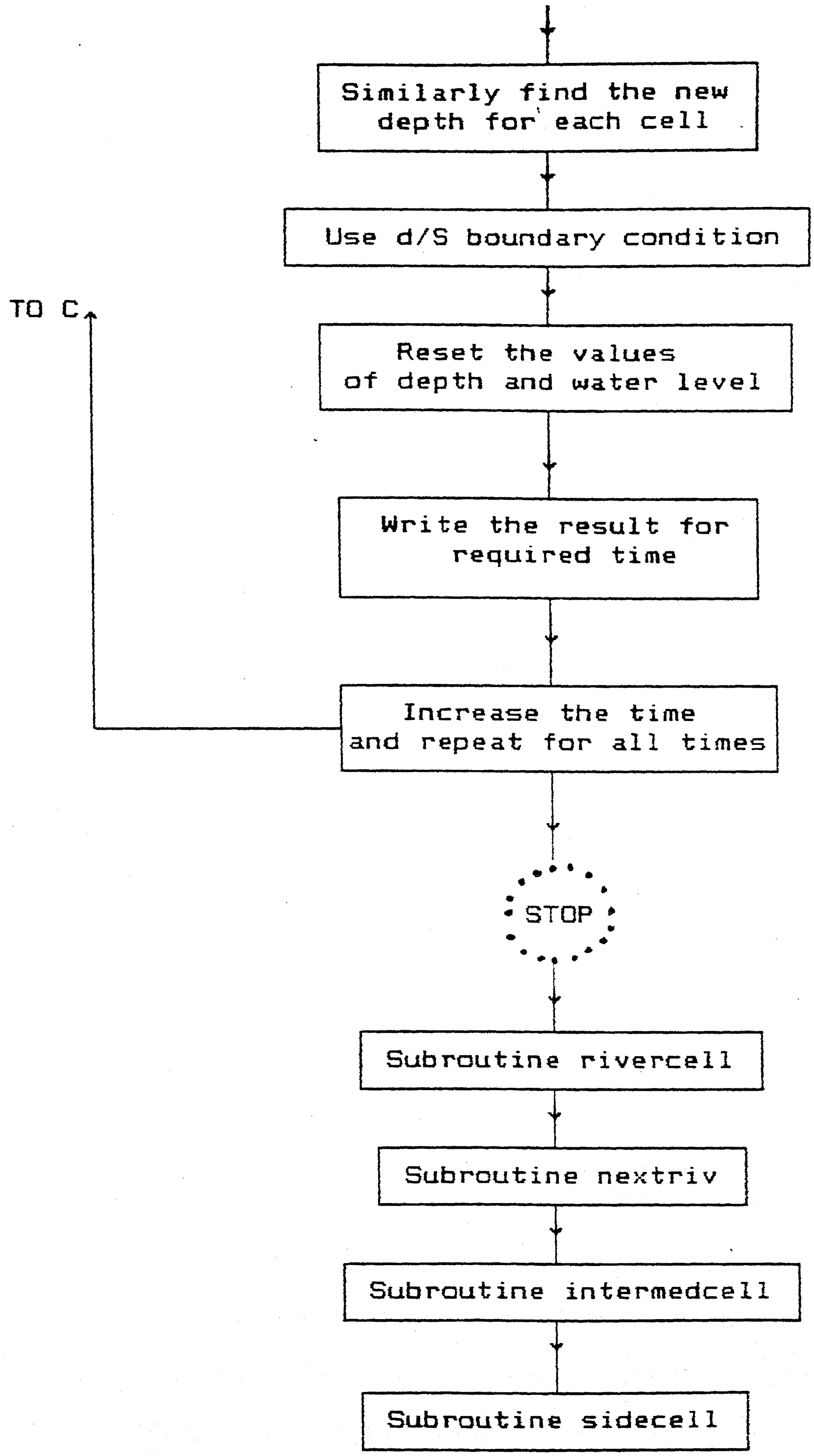
3.8 COMPUTER PROGRAM STRUCTURE

Explicit method for solution has been used for the present study. Runge Kutta method of 4th order has been used for the solving the ordinary differential equation. The flow chart is given later and the general program is given in Appendix A.

The main program calculates the initial conditions, boundary conditions and solves the ordinary differential equation by Runge Kutta's method. As mentioned earlier, there are four kinds of cells, namely, river cell, next to river flood plains cell, intermediate flood plains cell and side cell. Separate subroutines have been formulated for the calculation of exchanges of flow between cells for each kind of cell. The constant time step Δt of 2 sec. has been used. However, results are printed at intervals of 120 sec. The stage hydrographs are obtained at 2 km. intervals in main river as well as on flood plains. The peak stage and time of occurrence for those cells also have been obtained. Flow chart shows the algorithm used in the program and gives clear picture about it.

FLOW CHART


Read and write channel parameters
Set initial conditions
Set the u/S boundary condition to find the water level in u/S cells
Unsteady calculation at interior nodes starts
C
Identify the cell type
Call corresponding subroutine and find K_1,K_2,K_3 and K_4 in applying R.K. method and finally find dzw
Calculate new water level and water depth at the end of that time level as
Zw_n = Zw + dzw
Y_n = y + dZw


Similarly find the new depth for each cell
Use d/S boundary condition
TO C
Reset the values of depth and water level
Write the result for required time
Increase the time and repeat for all times
STOP
Subroutine rivercell
Subroutine nextriv
Subroutine intermedcell
Subroutine sidecell

3.9 COMPUTATIONAL DIFFICULTIES

In this section, computational difficulties, which were faced during developing the model and its remedies are discussed. The concern here is not with the formal mathematical behaviour of various numerical schemes but with the numerical treatment of particular situations related to the fact that we try to solve continuous flow system with a discrete model. One should bear in mind the fact the difficulties such as described below may not appear at all in certain modelling systems and on the contrary, other troubles may occur in such systems which are not mentioned here.

3.9.1 Weir Oscillation

In certain situations and for certain modelling systems which do not use an iterative solution procedure, the direction of flow over the Weir at a given time step can be in opposite sense from what it should be, and the persistence of the anomaly can destroy the calculation.

Fig. 3.4(a), (b) and (c) represent three consecutive time steps. In Fig. 3.4(a) flood overflows arrive first in cell A, so that its level rises above at the end of the time step. During the next time step, the program which had not used an iterative procedure calculates a discharge from A to B. But due to tremendous amount of water exchange or small surface area in B, its level jumps to the position shown in Fig. 3.4(b) at the end of the time step. Of course, iterative procedures would correct

it but if they are not used then at the beginning of third time step (b), the situation is anomalous in that the flow occurs in reverse direction i.e. from cell B to cell A and at the end of time step (C) again the water level in cell A goes higher than water level in cell B. This type of oscillation generally diminishes especially if each of two cells communicates with others. But if the cells are isolated from other, the oscillation can become unstable and destroy the calculation.

One solution to this kind of problem is to use iterative procedures to avoid oscillation, but it may take considerable computer time. Usually it is less costly to accept some inconsistency like the above oscillations provided that volume continuity is not violated beyond reasonable limits.

3.9.2 Different Alternative Methods Attempted and Problems Associated with Them

In the beginning the problem has attempted taking water surface slope in the Manning's equation for river links in the longitudinal direction as suggested by Cunge but the solution was found to be highly unstable. However, Cunge did not mention in his work about such kind of difficulties. Hence, instead of water surface slope, bed slope was used in the Manning's equation for river links in the longitudinal direction.

A number of different alternatives for upstream boundary condition were tried including the discharge inflow hydrograph instead of stage inflow hydrograph. In the case of discharge hydrograph, compound channel approach was used to determine the depths in the river and the flood banks at each time step. Unnecessarily lot of iterations were required for binding out the depths in the upstream cells, consequently consuming more computer time without improving the results. To make the computational procedure simpler, stage hydrograph was taken as upstream boundary condition.

3.10 SELECTION OF INPUT-DATA

A symmetric prismatic channel with flood plains as shown in Fig. 3.1 has been considered for the study. The length of the channel has been taken 10 km but only first 8 km has been studied to avoid the effect of down stream boundary. According to Fig. 3.1, the main channel width B_m is equal to 50 m and the bank full depth is equal to 5 m. the manning's n of main channel has been taken as 0.03. The manning's n of the flood plain is varied from 0.03 to 0.12. The slopes of the main channel and the flood plains are same and equal to 0.0005. the initial flow depth in main channel is 5.5 m and in flood plains 0.5 m. The datum line is assumed to be horizontal and it is 10 m below the upstream

main river cell. The duration of stage hydrograph is 1 hr. and total time of computation is 8 hr.

The relationship between the dependent variables and the independent variables of the problem under study are as follows:

$$y,\ y',\ y_p = f_{1,2,3}\ (x, t, B_m,\ B_s,\ n_1,\ n_2,\ t_{peak})$$

where,

y = water level in the river cells above river bed level at section under consideration.

y' = water level in the flood plain cells above river bed level at that section.

y_p = peak depth over a cell under consideration.

x = distance of cell from u/s boundary.

t = time of computation

B_m = width of main channel.

B_s = total width of channel.

n_1 = manning's n of main river.

n_2 = manning's n of flood plains.

t_{peak} = time up to peak stage of inflow hydrograph.

B_m, B_s, n_1 and n_2 are characteristics for the channel and the flood banks, t_{peak} is characteristics of the inflow hydrograph.

Dimensional analysis of the above listed variables will lead to the following functional relationships between the dimensionless groups:

$$\frac{y}{y_0}, \frac{y'}{y_0}, \frac{y_p}{y_0} = f_{5,6,7} \left(\frac{x}{y_0}, \frac{t}{t_{peak}}, B_r, n_r \right)$$

$$\frac{t_p}{t_{peak}} = f_8 \left(\frac{x}{y_0}, B_r, n_r \right)$$

where,

$$B_r = \frac{B_s - B_m}{B_m} \quad \text{and}$$

$$n_r = \frac{n_2}{n_1}$$

The effect of variation of B_r and n_r was studied and all the other parameters were kept constant. The above said parameters were varied one by one keeping others constant to see the effect of the varied parameters.

3.11 COMPUTATIONS

The governing equations were solved by explicit method and further for solving ordinary differential equation Runge Kutta's fourth order method was applied. The whole channel reach was divided into 20 cells along length with each cell of length 500 m. There was only one cell along transverse direction in main channel. For the value of B_r equal to 4, the number of rows of cell over flood plains was 4 whereas for B_r = 8 it was 8. The width of each cell was 50 m. the value of time step was taken equal to 2 sec. There were four sets of data as shown below.

Set 1:	$B_r = 4$	$n_r = 1.0$
Set 2:	$B_r = 4$	$n_r = 4.0$
Set 3:	$B_r = 8$	$n_r = 1.0$
Set 4:	$B_r = 8$	$n_r = 4.0$

In all the above cases unless otherwise specified the values of other parameters were taken as follows:

$dl = 50$ m

$y_f = 5$ m

$d_x = 500$ m

$S_o = 0.0005$

$y_0 = 5.5$ m

$Z_{wo} = 15.5$ m

$Z_{wop} = 18.0$

$n_1 = 0.03$

$g = 9.81\ m^2/sec$

$t_{peak} = 1800$ sec

$t_b = 3600$ sec.

$t_{last} = 28800$ sec.

3.12 RESULTS AND DISCUSSIONS

As mentioned earlier, four cases have been studied for different combinations of B_r and n_r. The results of the numerical analysis are shown in figures. Figs. 3.5 through 3.8 show the effect of n_r and B_r on the stage hydrographs at different locations at 2 km, 4 km, 6 km and 8 km from u/s boundary. Inflow stage hydrograph at u/s boundary has been included in each case. Fig. 3.5(a) through Fig. 3.8(a) show the stage hydrographs along river cells whereas Fig. 3.5(b) through Fig. 3.8(b) show for flood plain cells. Fig. 3.5(c) through Fig. 3.8(c) show the stage hydrographs at same locations for one-dimensional dynamic model developed by Mahapatra, K. (1990). The plots have been made with normalized time on x-axis and normalized depth on y-axis. The figures 3.5 and Fig. 3.7 reveal that in the case of two dimensional model there is no subsidence in peak depth for the value of $n_r = 1.0$ where as there is considerable subsidence in peak depth in the case of

one-dimensional dynamic model. Two- dimensional model has behaved like a kinematic model and inflow hydrograph has just shifted to next position for the value of $n_r = 1.0$. At the same time there is not much difference in the shape of stage hydrographs for flood plain cells and river cells. Figs. 3.6 and 3.8 reveal that there is more subsidence in one-dimensional dynamic model compared to two dimensional model even for the value of $n_r = 4.0$. However, in the cäe of $n_r = 4.0$, there is subsidence in two-dimensional model too.

If one compares the effect of B_r, keeping the constant value of n_r, it is found that there is little effect of B_r on stage hydrographs in two-dimensional model. When the value of n_r is equal to 4.0 the time taken to reach the peak depth at any location is more in two-dimensional model compared to one dimensional model at the same location. This is due to ignoring dynamic effect. The one-dimensional dynamic model has higher wave velocity compared to two-dimensional model without inertial effect.

As it is well known fact that celerity along river is much higher than along flood plains, even then there is no difference in the shape of stage hydrographs in both of them. This is because in river type link equation along longitudinal direction bed slope rather than water surface slope has been used.

Figs. 3.5(a) through Fig. 3.8(a) and Fig. 3.5(b) through 3.8(b) show wavy fluctuations at the end of each stage hydrograph

which diminish with time and tend to steady state condition. This is due to down stream end effect, because down stream end is very near where as in one dimensional model down stream end is far away.

Fig.3.9(a) and 3.9(b) are presented to show the flood peak travel time with distance along the river cells and flood plain cells, respectively. The plots have been made with normalized location on x-axis and normalized peak time on y-axis. The figures reveal that n_r has more effect than B_r on flood peak travel time. It has been also noticed that with larger value of B_r, the effect is still more.

Figs. 3.10 show the variation of flood peak depth with normalized location. It is clear from Fig. 3.10(a) and Fig. 3.10(b) for the value of n_r = 4.0 there is considerable subsidence at the downstream locations. In cae of n_r = 1.0 they do not behave well and show fluctuation in peak depths in two-dimensional kinematic model. However in case of one-dimensional dynamic model for all the cases it behaves well. In one dimensional dynamic model as shown in Fig. 3.10(c) B_r has more effect than n_r on subsidence of peak along channel length. When the manning's n for flood plains has been made four times the 'n' for main river the peak depth ratio decreases faster i.e. the flood plains do play an effective role on flood subsidence for both the models. It is also observed that its effect is still more with larger value of B_r.

CHAPTER IV

CONCLUSIONS

4.1 CONCLUSIONS

The various conclusions made are as follows:

1. The result s of the two-dimensional explicit model for flood routine using only continuity principle differ to a great extent from the results of the one-dimensional explicit model using both continuity equation and equation of motion. Some of the results of the two-dimensional model exhibit anomalies also.
2. The two-dimensional model with $n_r = 1.0$ does not exhibit any flood peak subsidence as the flood wave propagates down the river. Whereas the model with $n_r = 4.0$ exhibits flood peak subsidence. The stage hydrographs exhibit oscillations in the recession side of the hydrograph.
3. The time taken to reach peak depth at different locations with $n_r = 4.0$ in the cae of two-dimensional model is more than the time taken in one-dimensional model.
4. In the two-dimensional model there is not much of a difference in the shape of the stage hydrographs of the river cells and the flood plain cells at any section for a given set of data.

5. The variation of peak depth with distance along the direction of flow exhibits oscillations for n_r = 1.0 where as the variation does not exhibit oscillations for n_r = 4.0.
6. The variation of time to reach peak depth with distance along direction of flow shows that the effect of n_r is more compared to B_r.

4.2 RECOMMENDATIONS FOR FUTURE WORK

The anomalies occuring in the present study are due to the explicit method used to solve the governing equations. So the same case should be studied using implicit method.

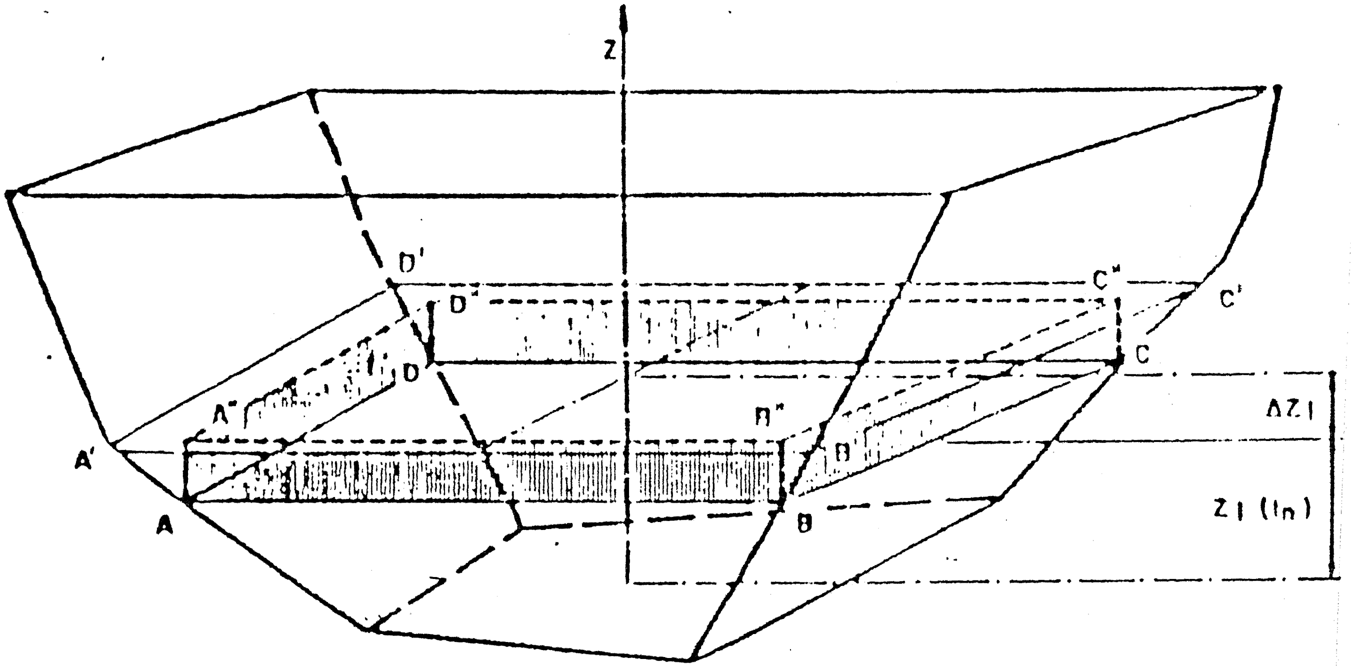


Figure 2.1 Continuity equation of a cell.

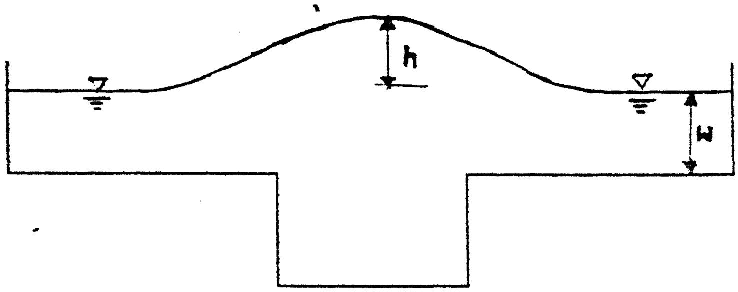


Fig. 2.2(a) (During rising Flood)

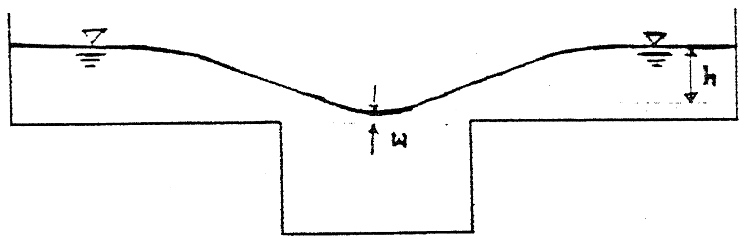


Fig. 2.2(b) (During falling Flood)

Weir type link

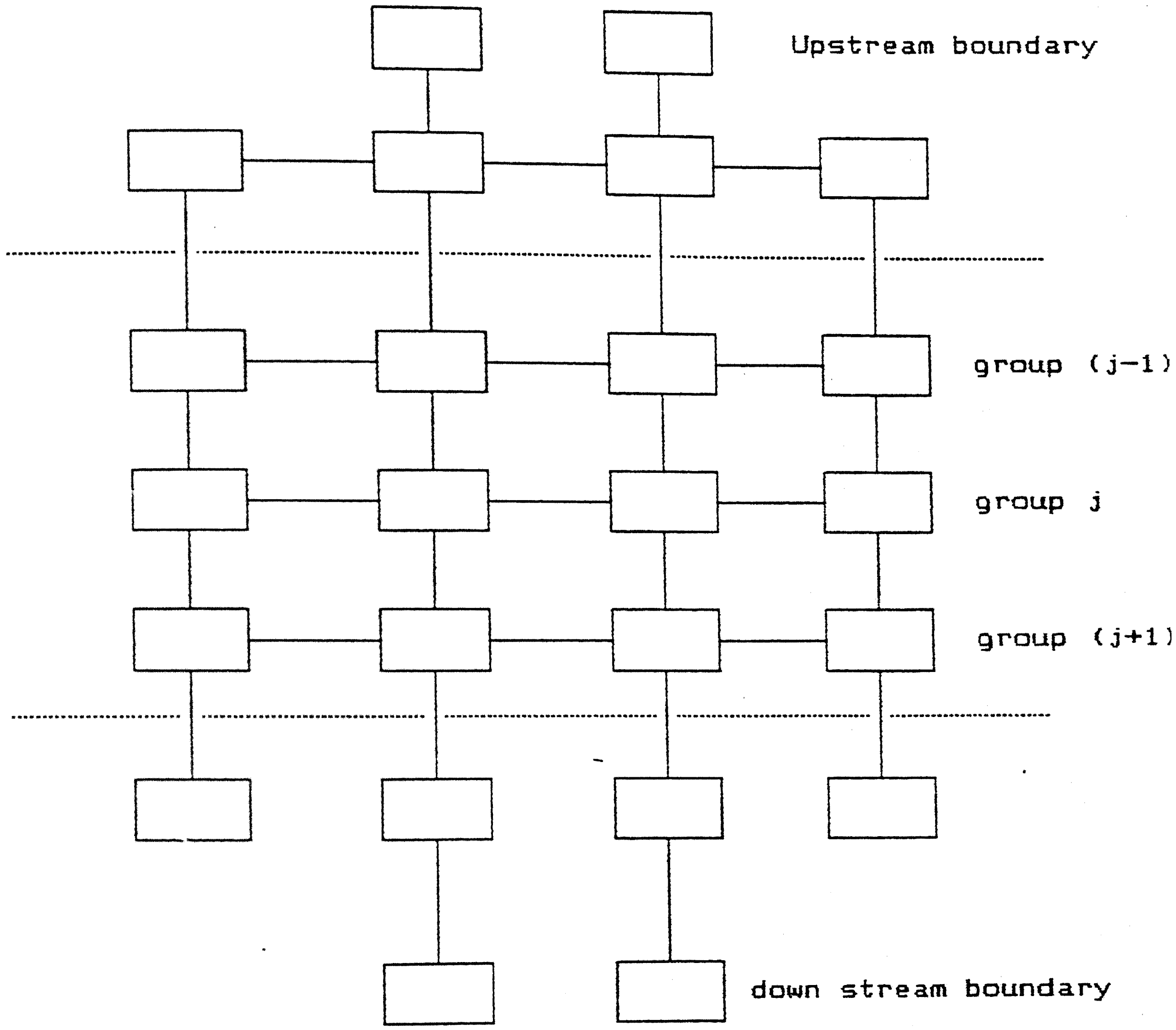


Fig. 2.3 Arrangement of cells groups

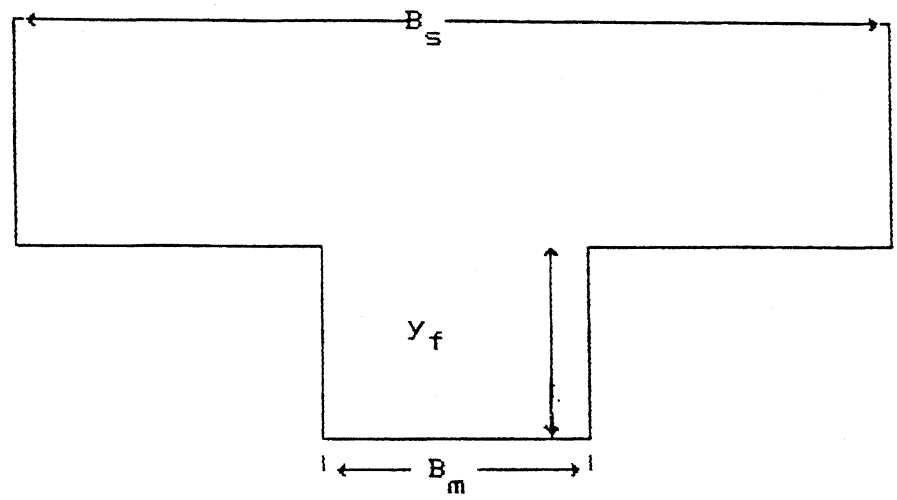


Fig. 3.1 Channel Cross-section with flood plains

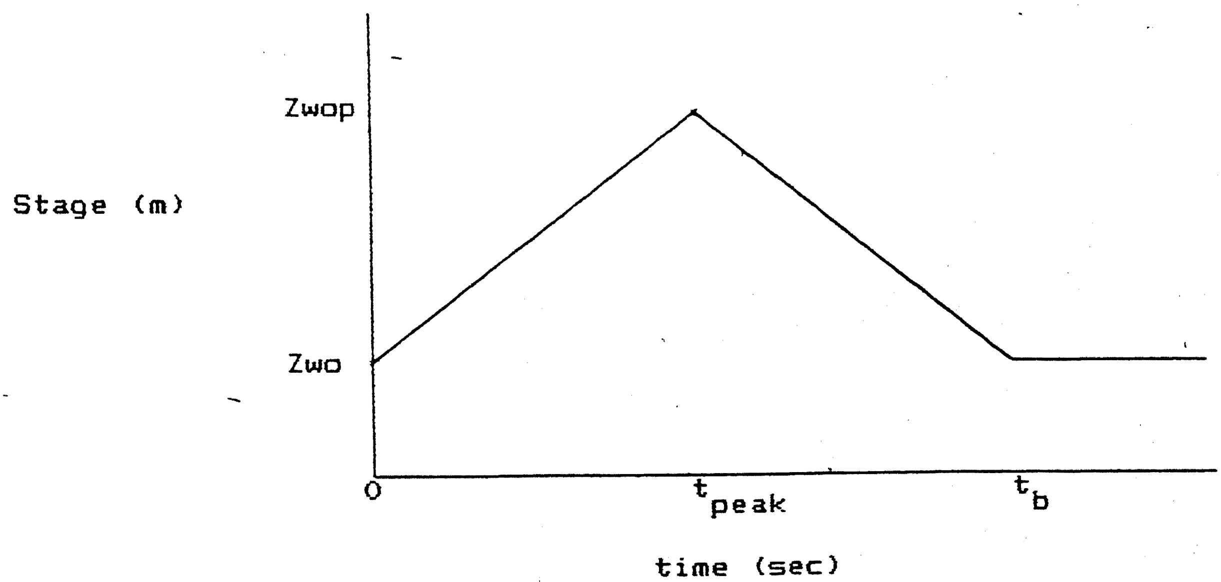


Fig. 3.2 Inflow hydrograph

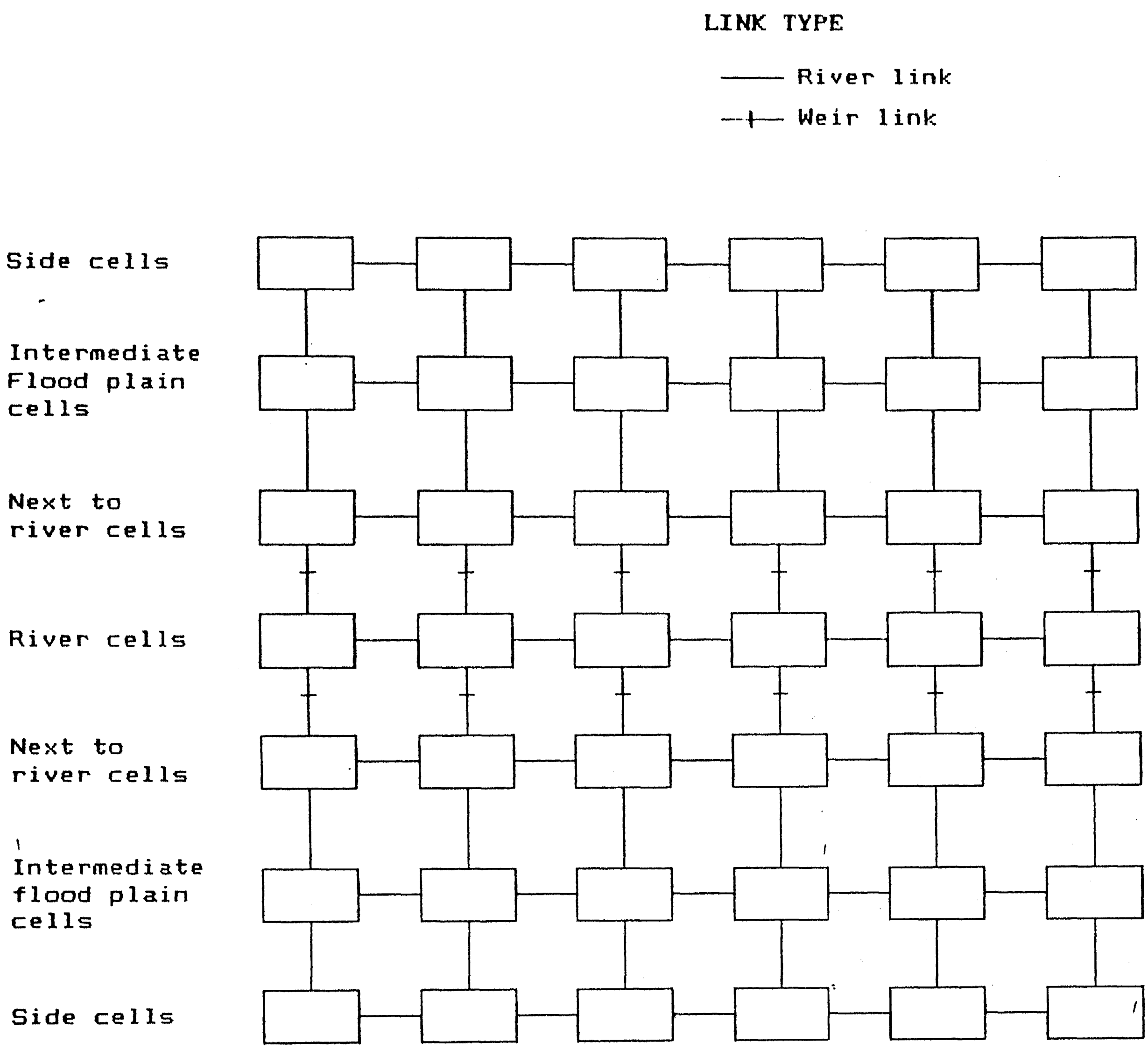


Fig. 3.3 Topological breakdown of computational cells.

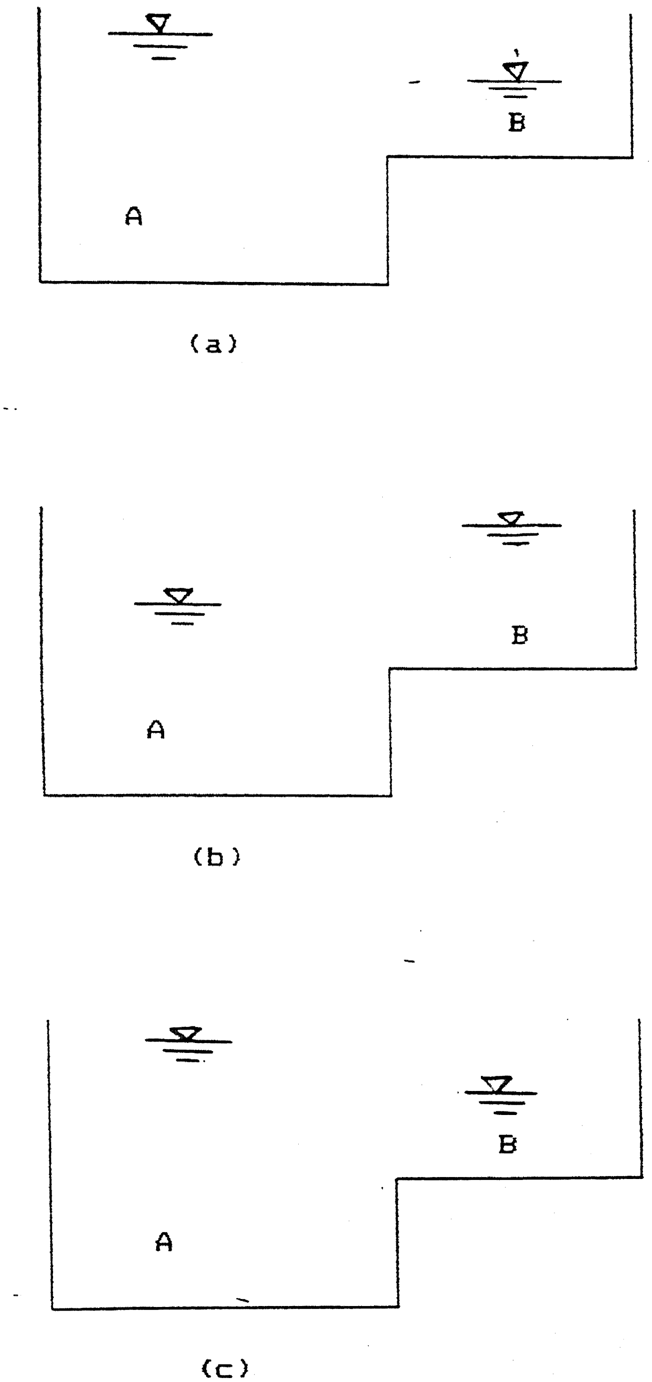


Fig. 3.4 Illustration of oscillations in Weir flow between two cells.

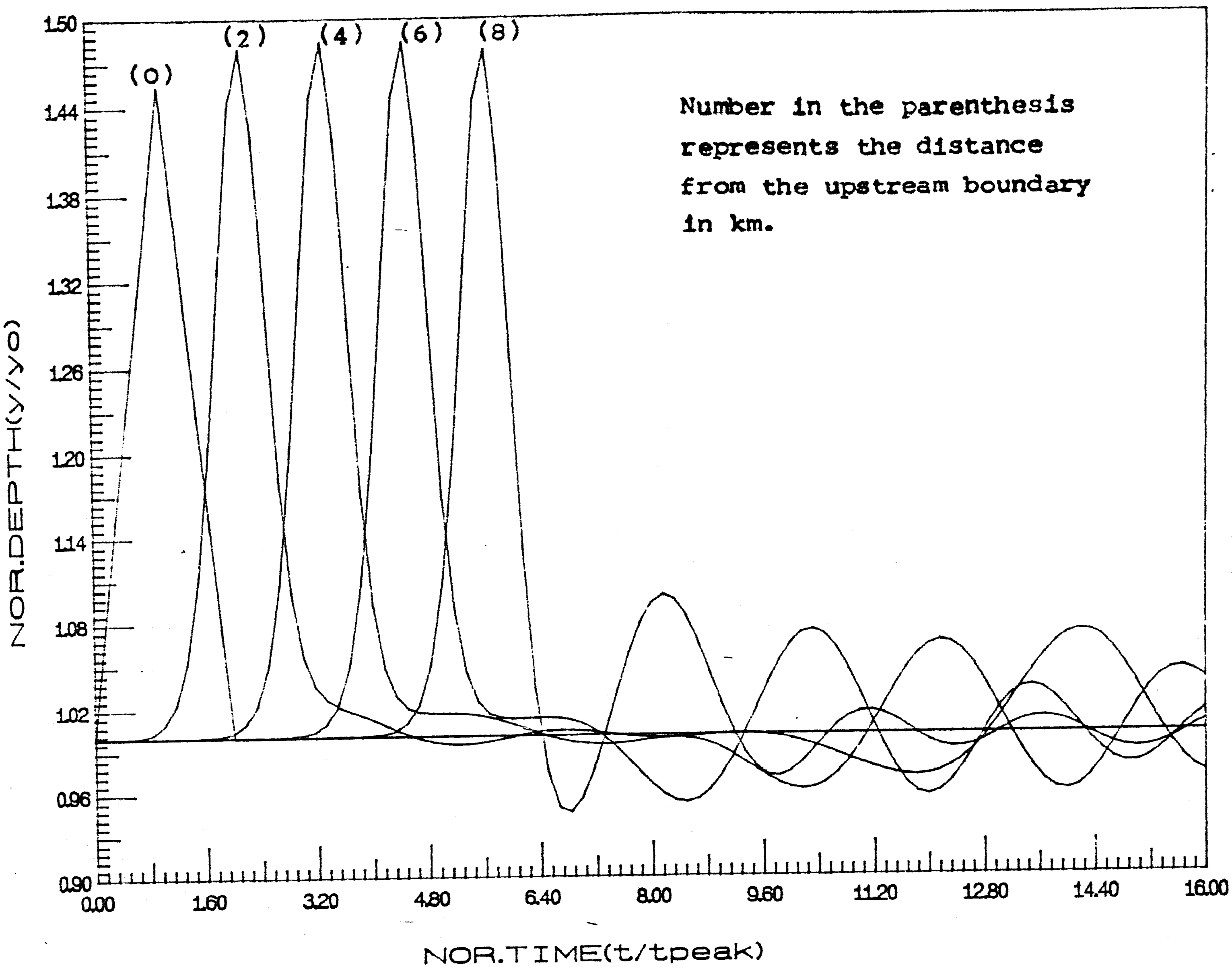


Fig. 3.5 (a) Stage hydrographs in main river for $B_r = 4.0$, $n_r = 1.0$ (2-D)

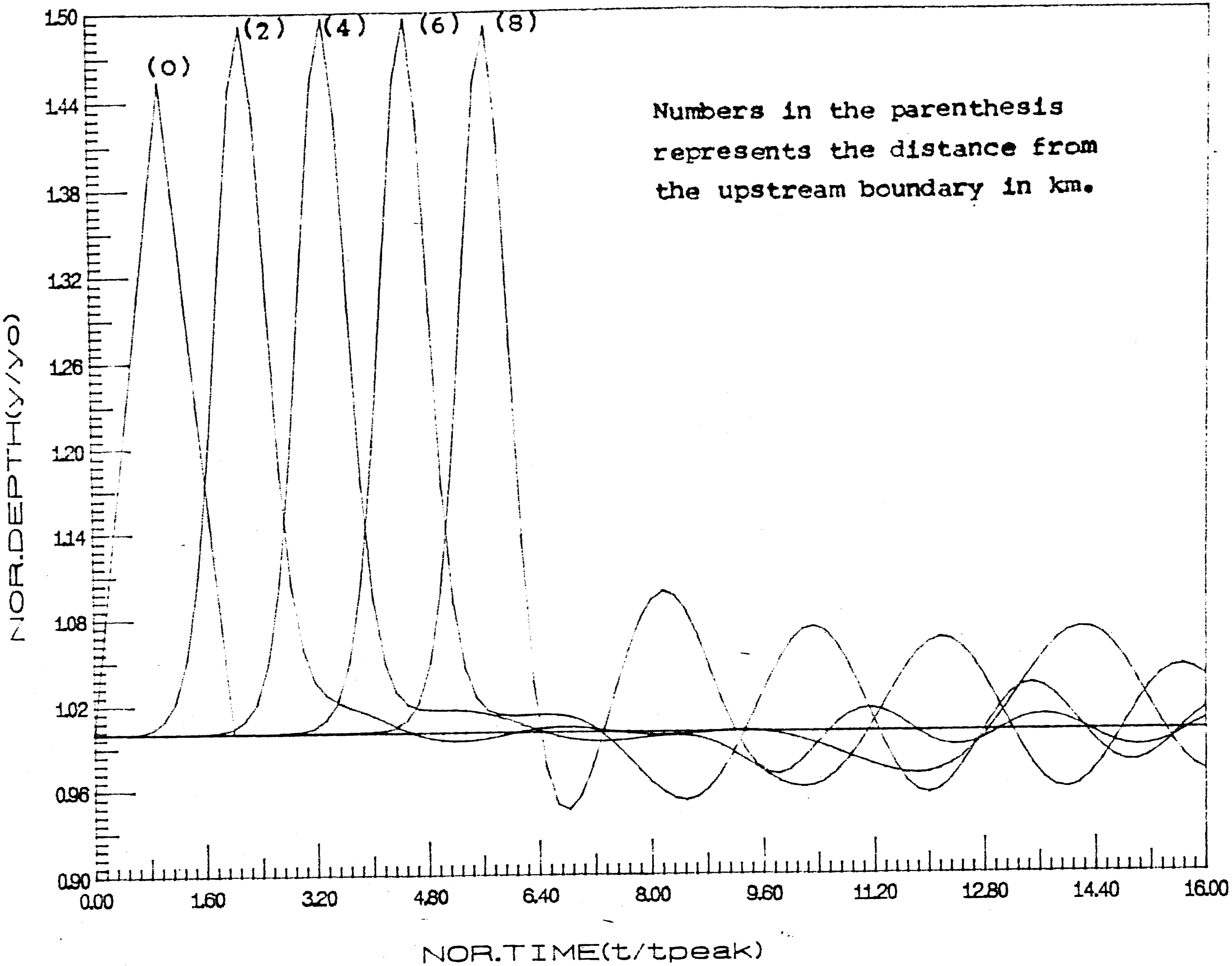


Fig. 3.5 (b) Stage hydrographs in flood plain for $B_r = 4.0$, $n_r = 1.0$ (2-D)

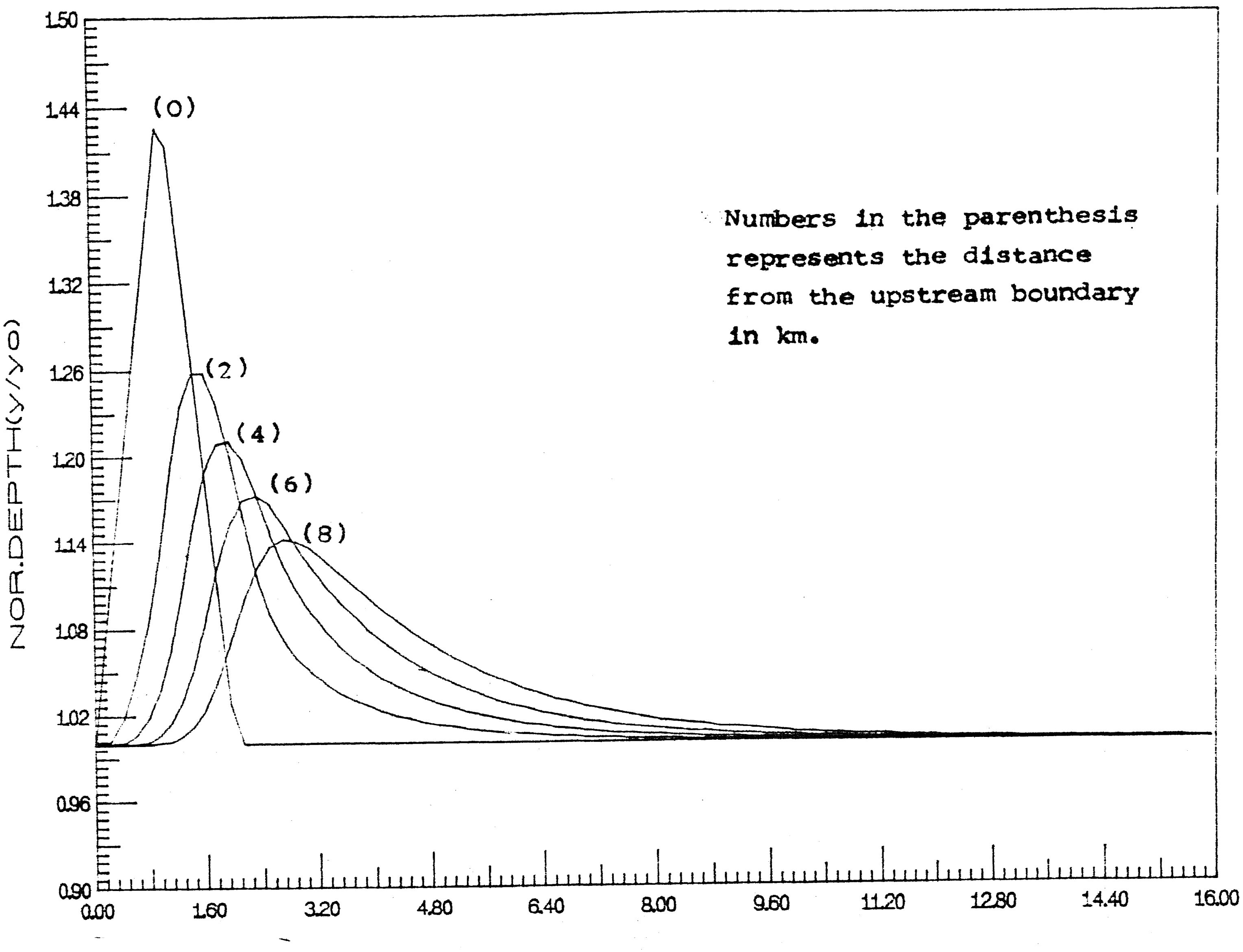


Fig. 3.5(c) Stage hydrographs for one - dimensional/for $B_r = 4.0$, $n_r = 1.0$ model

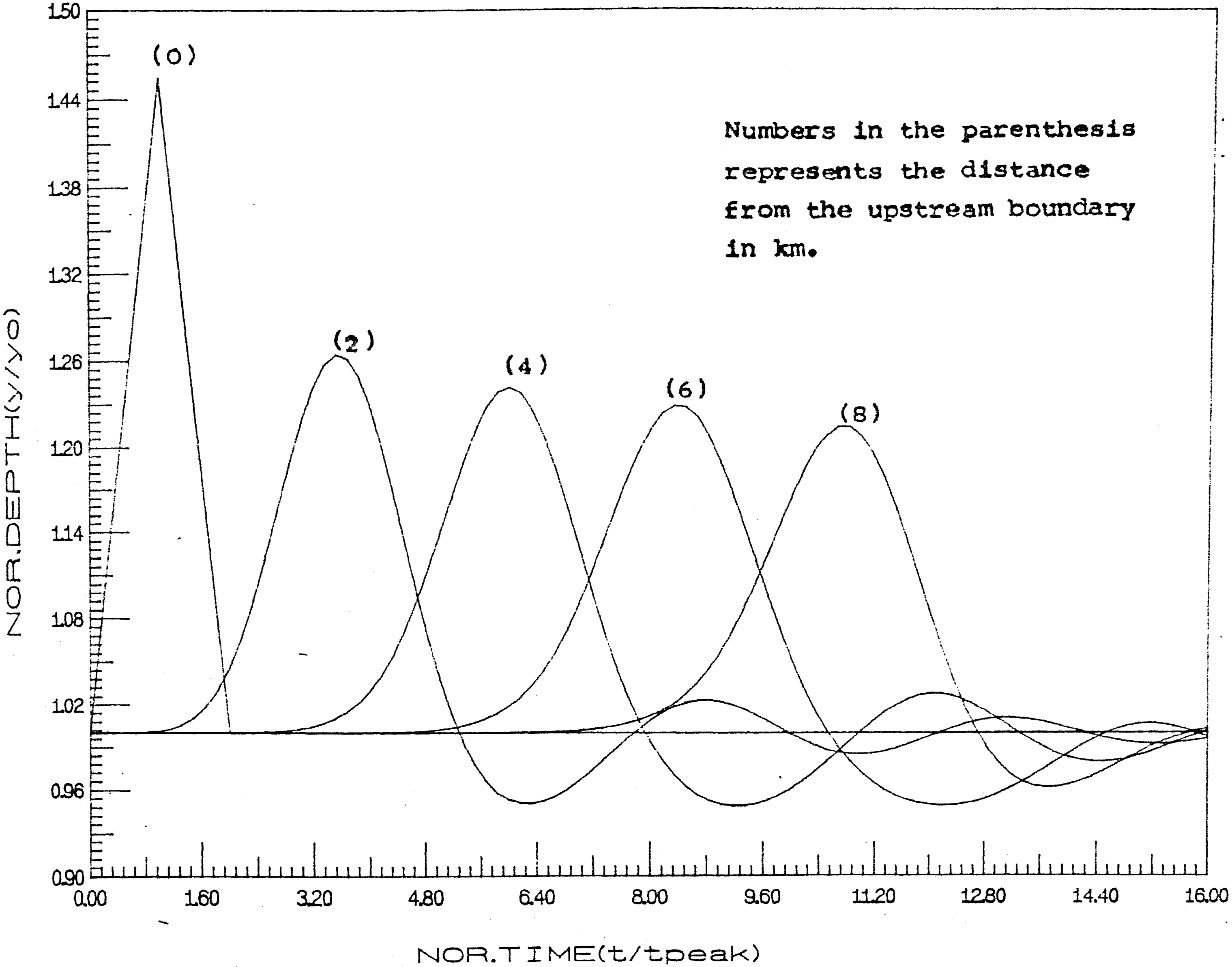


Fig. 3.6(a) Stage hydrographs in main river for $B_r = 4.0$, $n_r = 4.0$. (2-D)

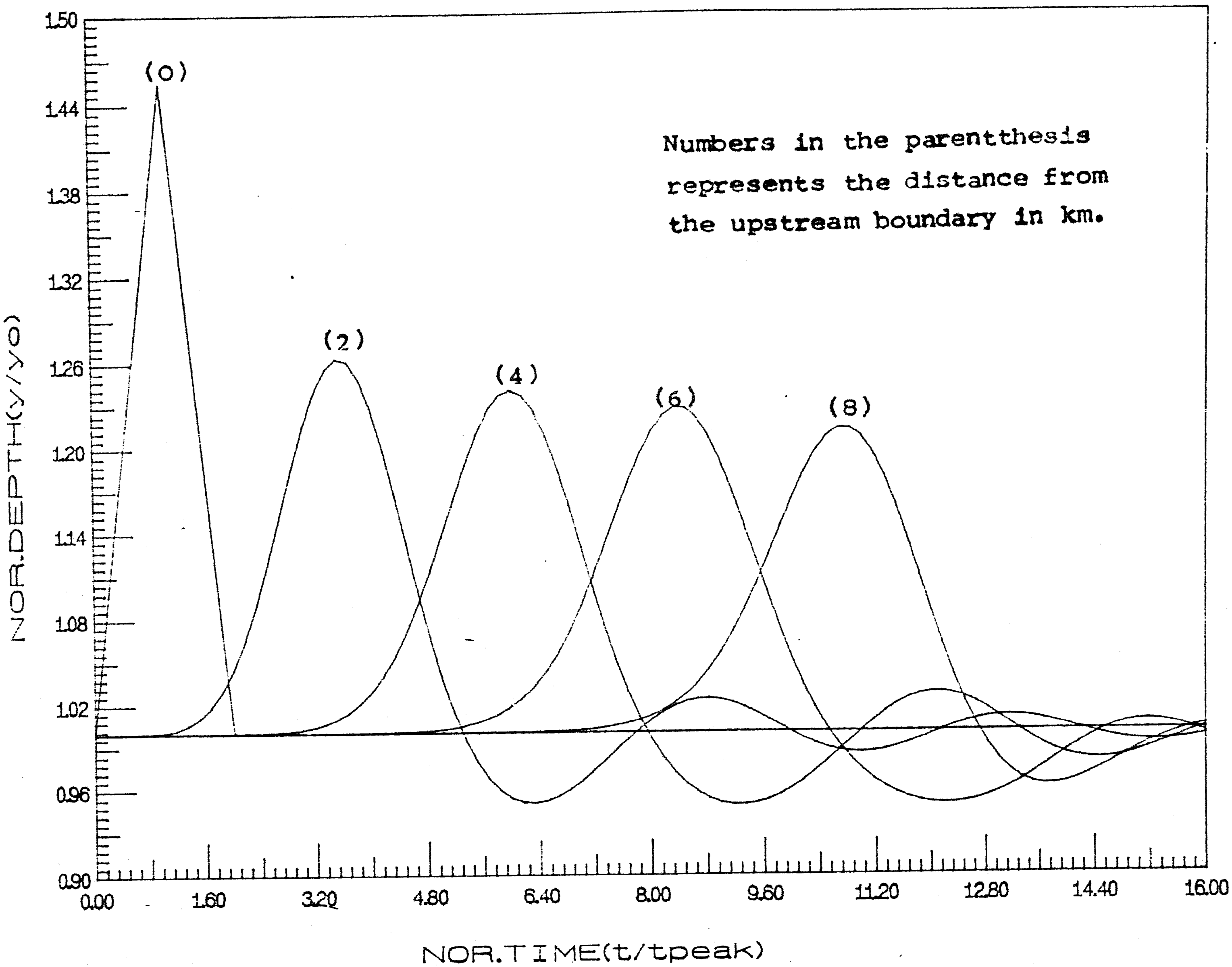


Fig. 3.6(b) Stage hydrographs in flood plain for B_r = 4.0, n_r = 4.0 (2-D)

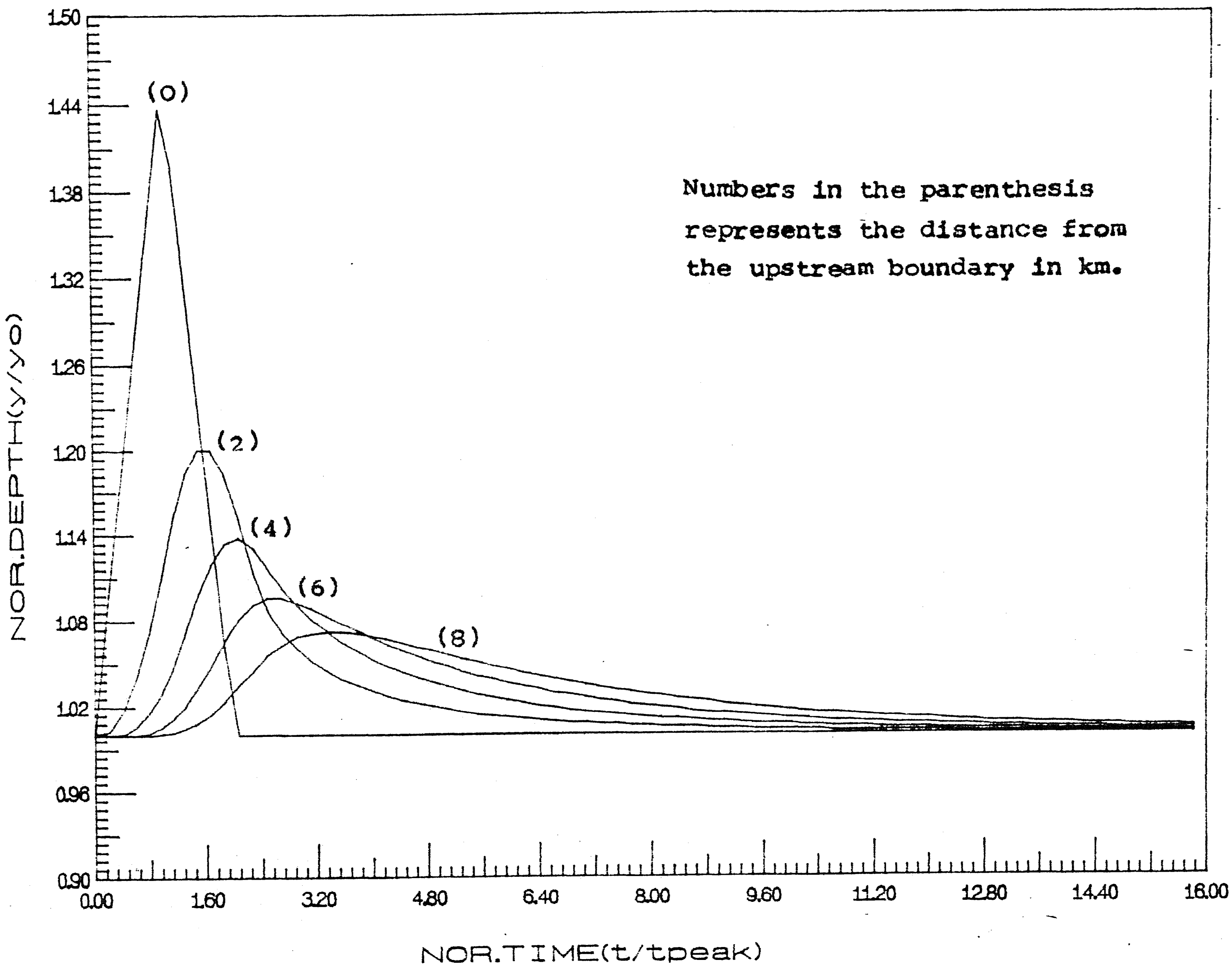


Fig. 3.6(c) Stage hydrographs for one dimensional model for $B_r = 4.0$, $n_r = 4.0$

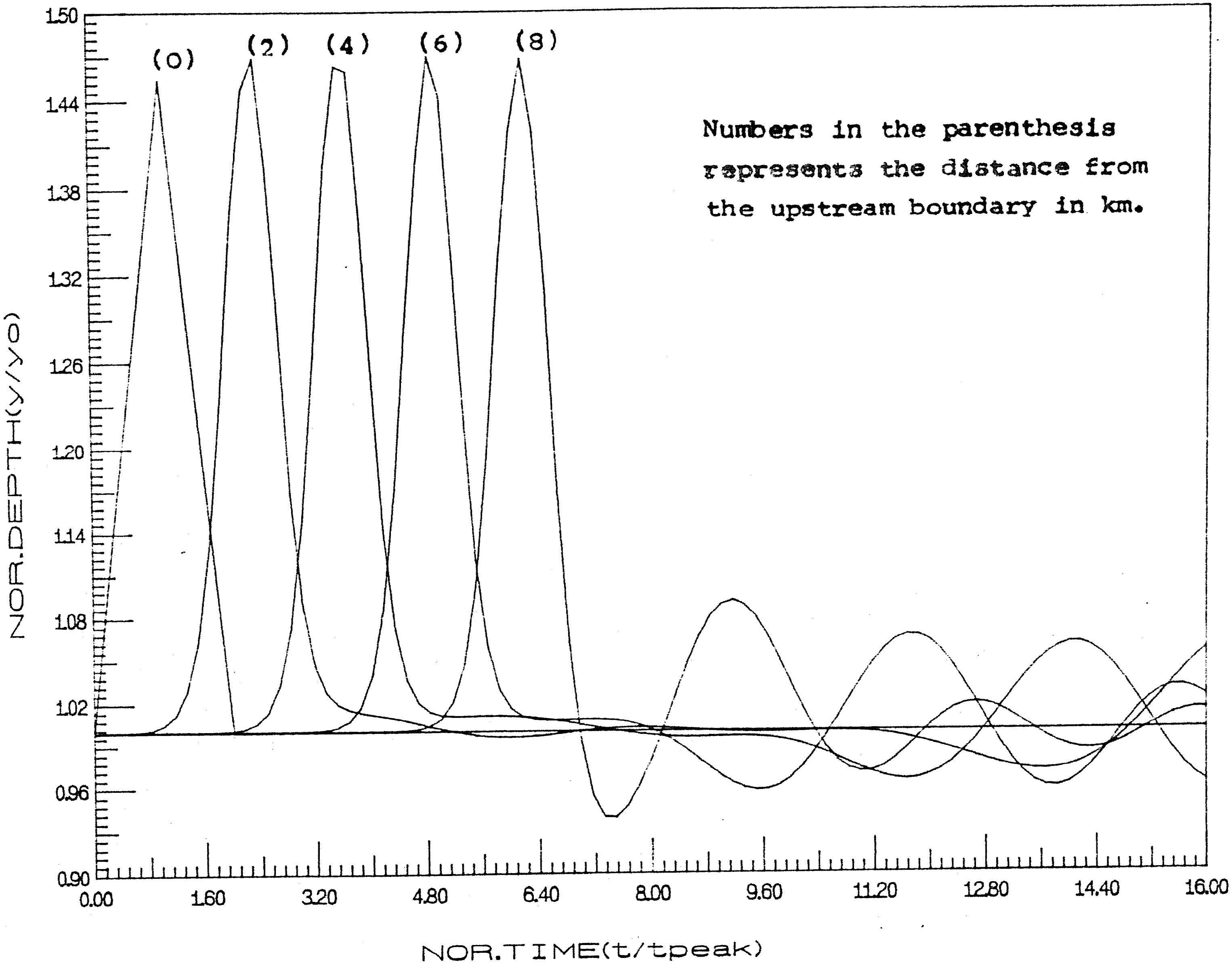


Fig. 3.7(a) Stage hydrographs in main river for $B_r = 8.0$, $n_r = 1.0$ (2-D)

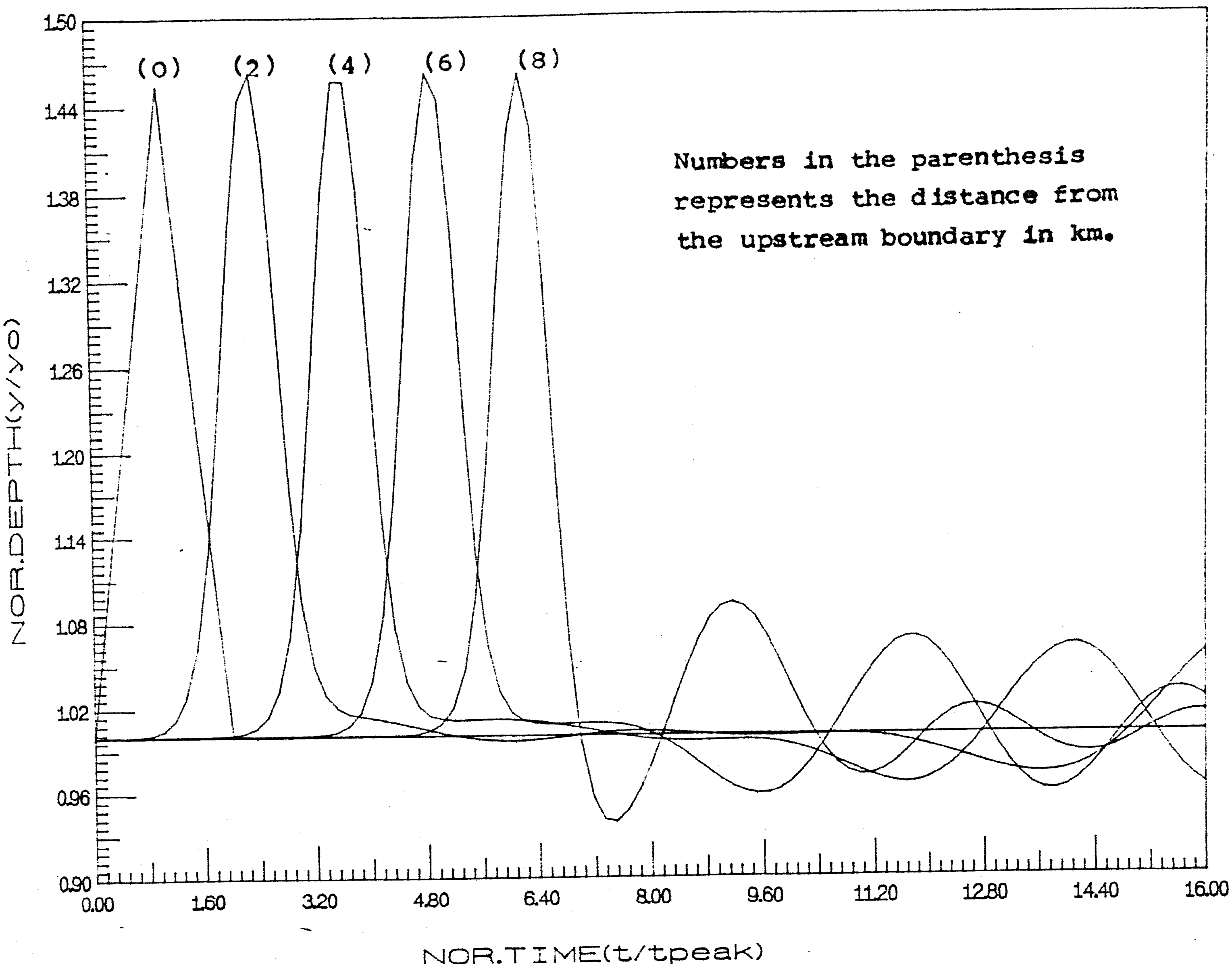


Fig. 3.7(b) Stage hydrographs in flood plain for $B_r = 8.0$, $n_r = 1.0$(2-D)

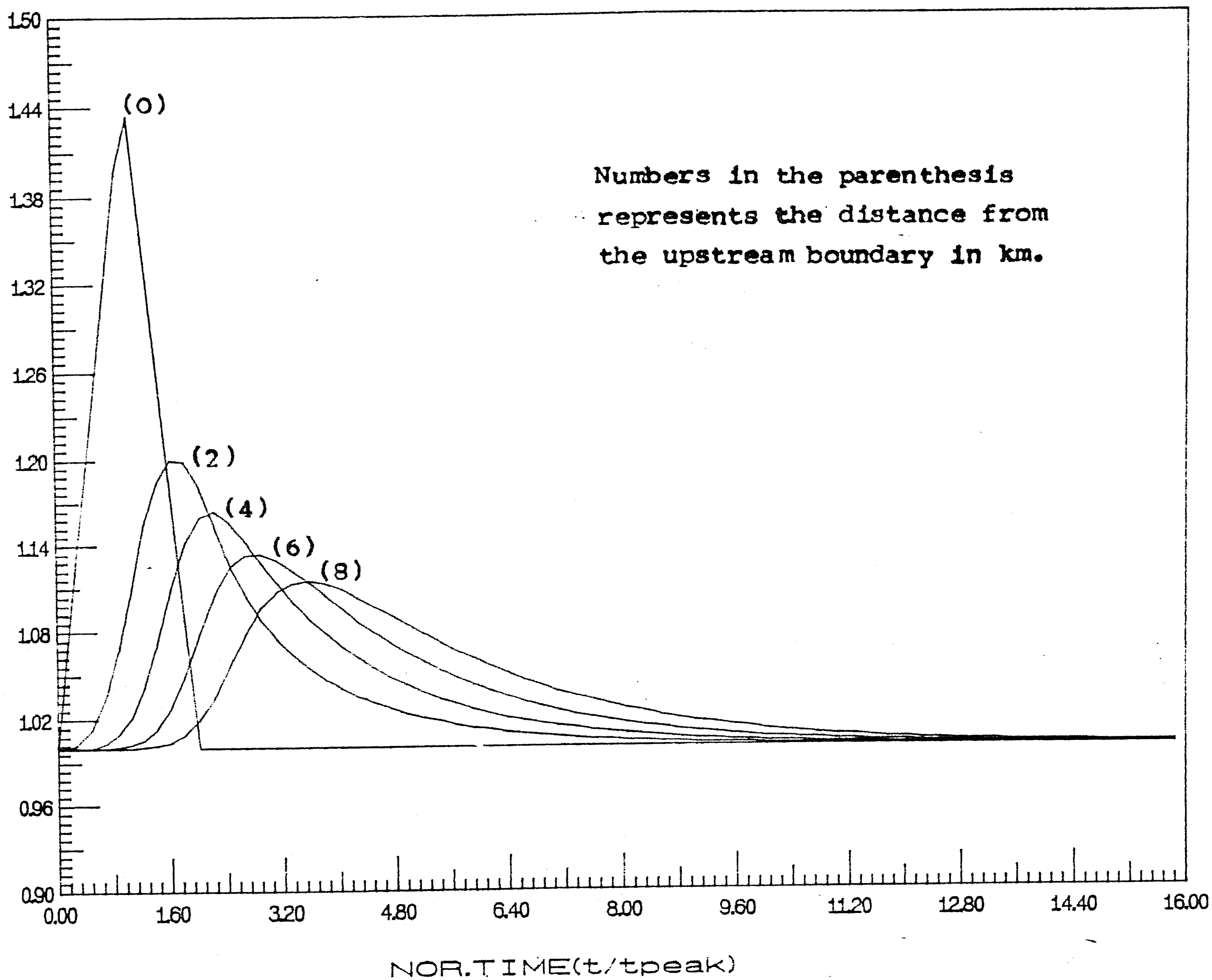


Fig. 3.7(c) Stage hydrographs for one dimensional model for $B_r = 8.0$, $n_r = 1.0$

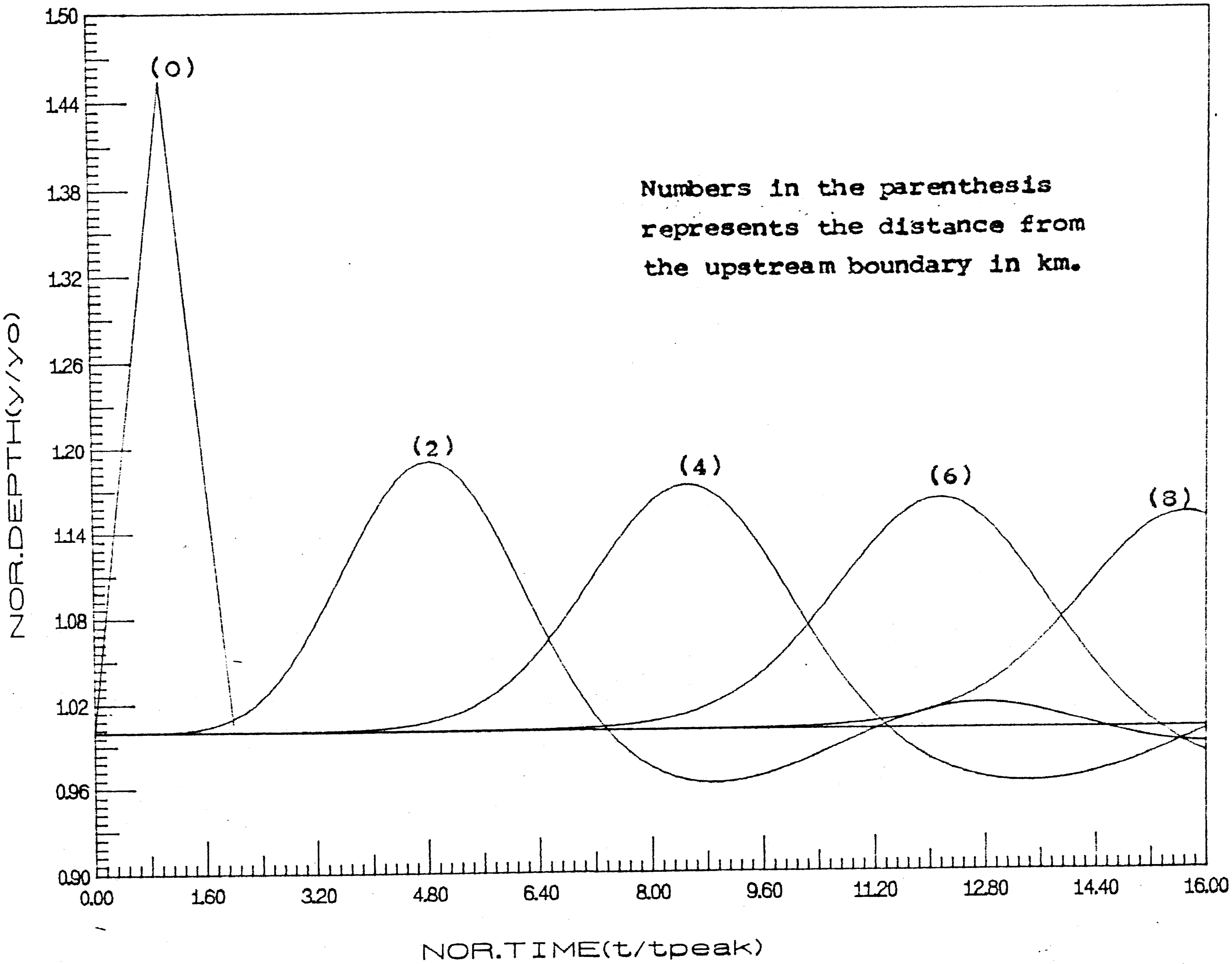


Fig. 3.8(a) Stage hydrographs in main river for $B_r = 8.0$, $n_r = 4.0$ (2-D)

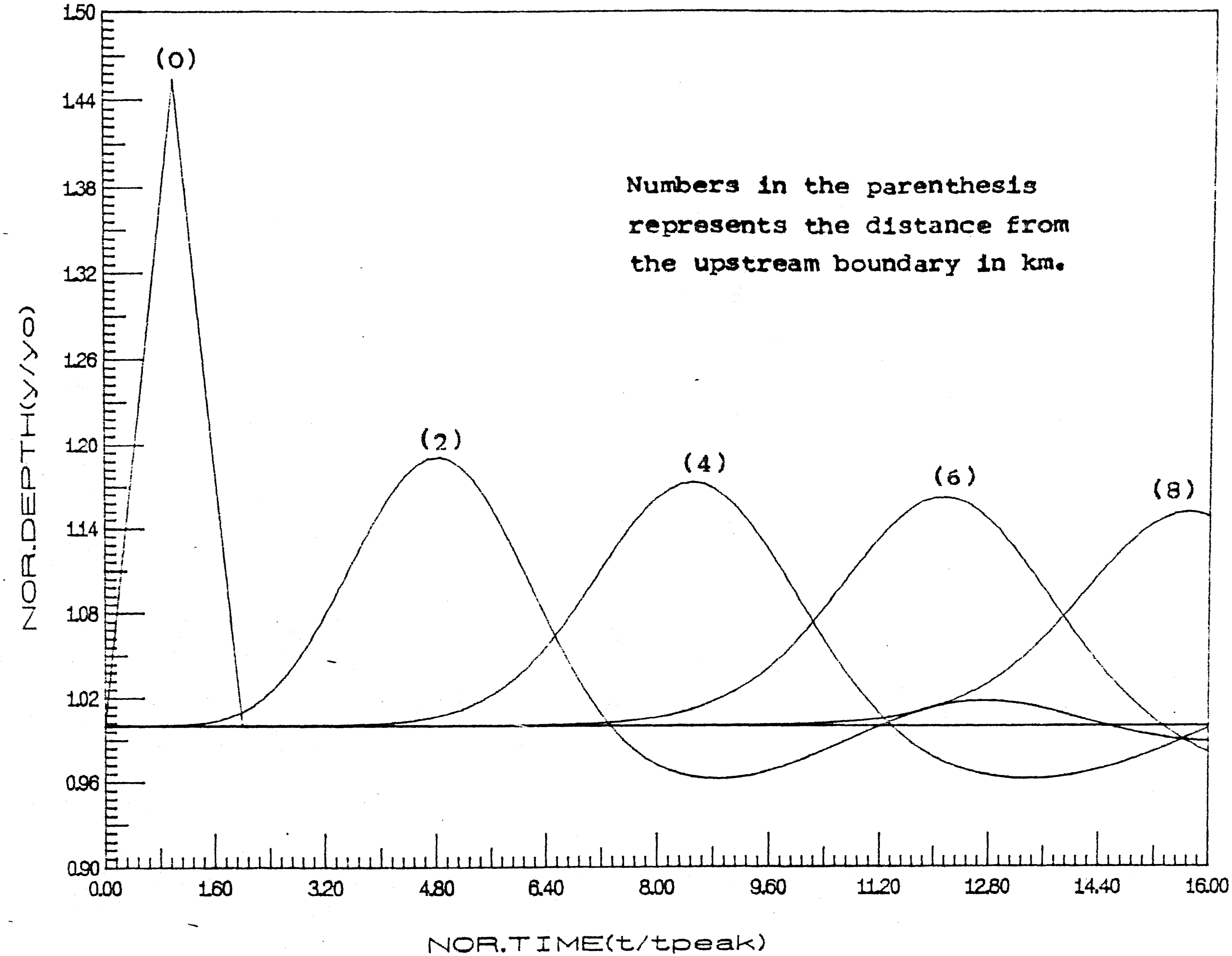


Fig. 3.8(b) Stage hydrographs in flood plain for $B_r = 8.0$, $n_r = 4.0$ (2-D)

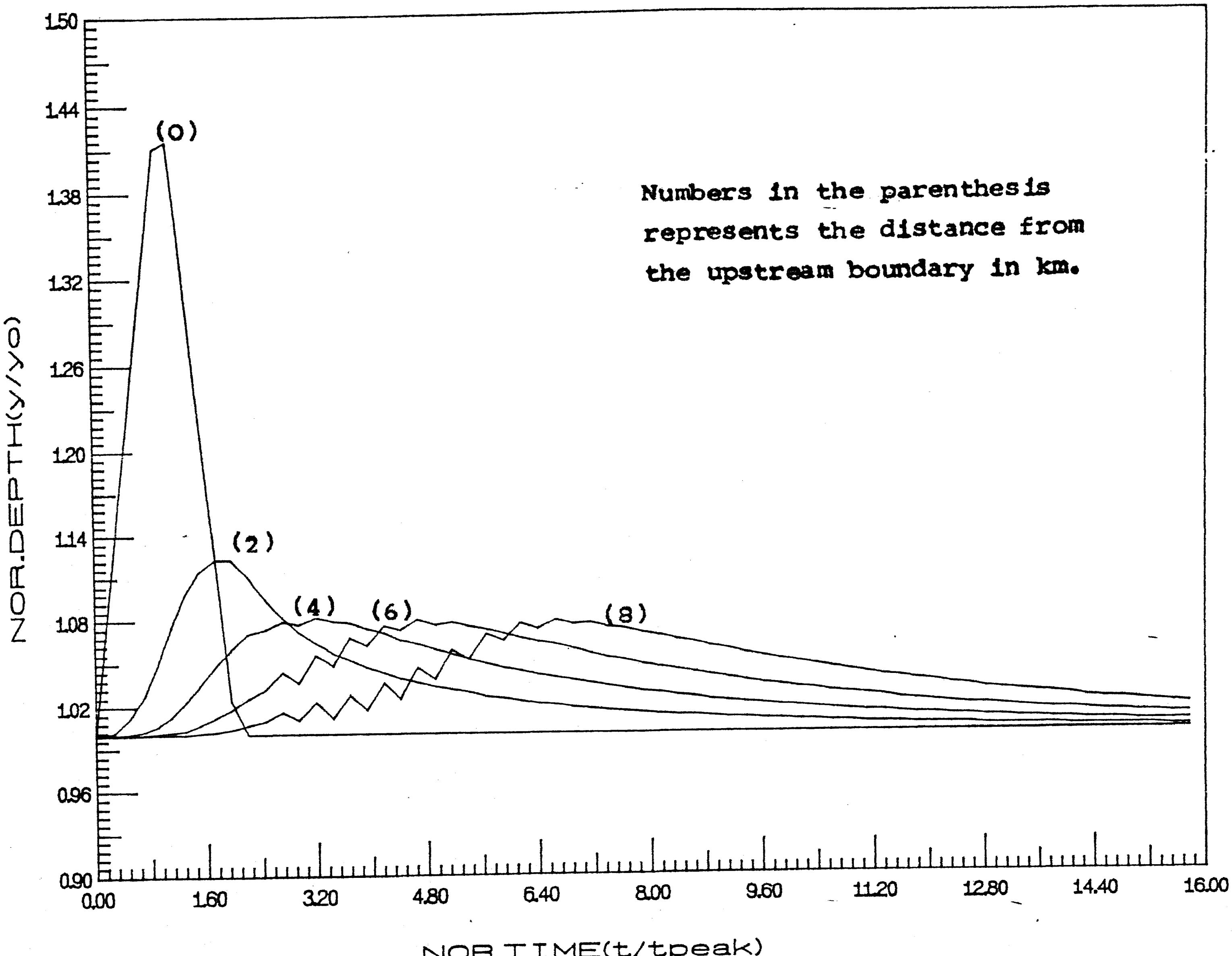


Fig. 3.8(c) Stage hydrographs for one dimensional model for $B_r = 8.0$, $n_r = 4.0$

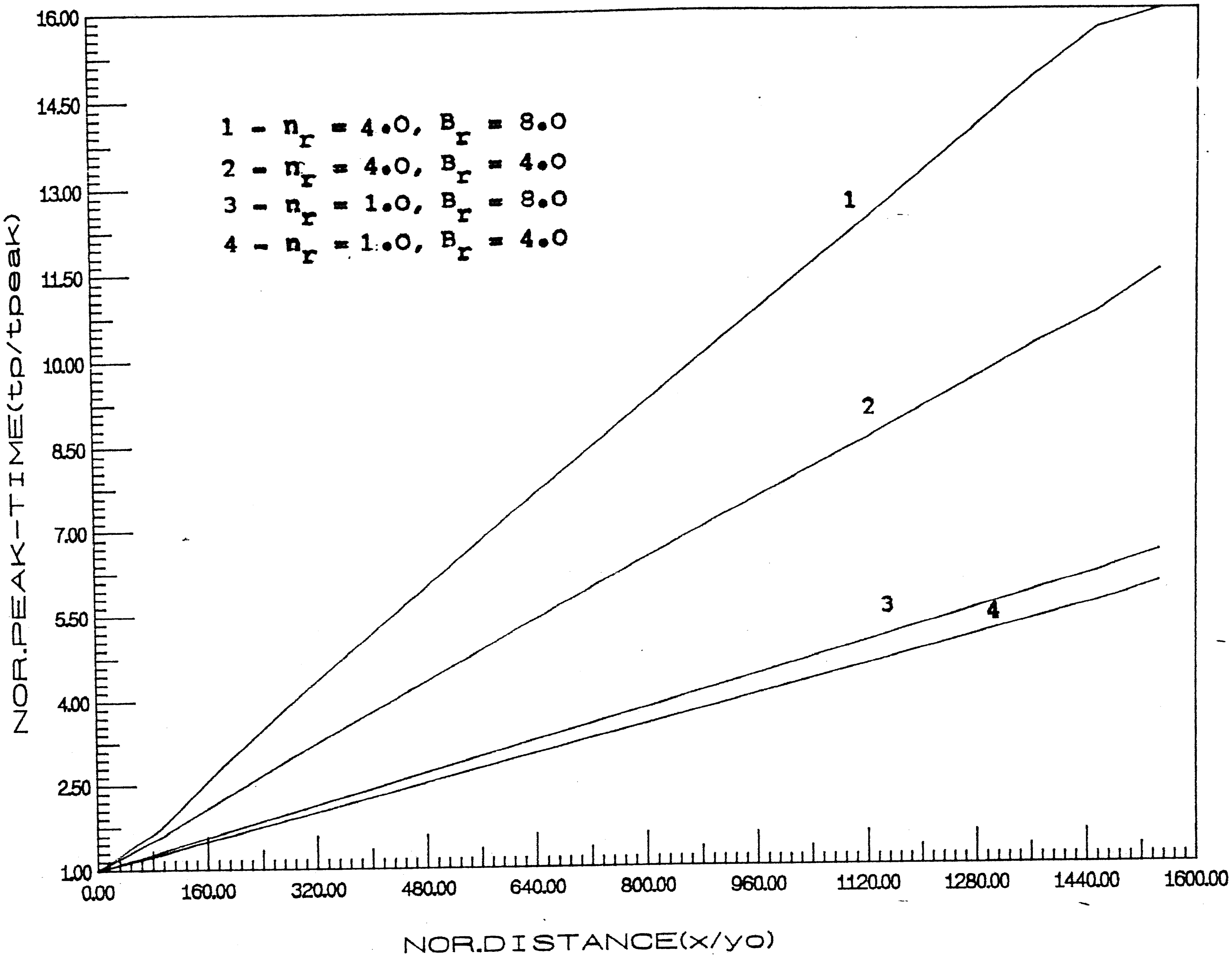


Fig. 3.9(a) Normalized time of peak depth vs normalized distance along the main river.

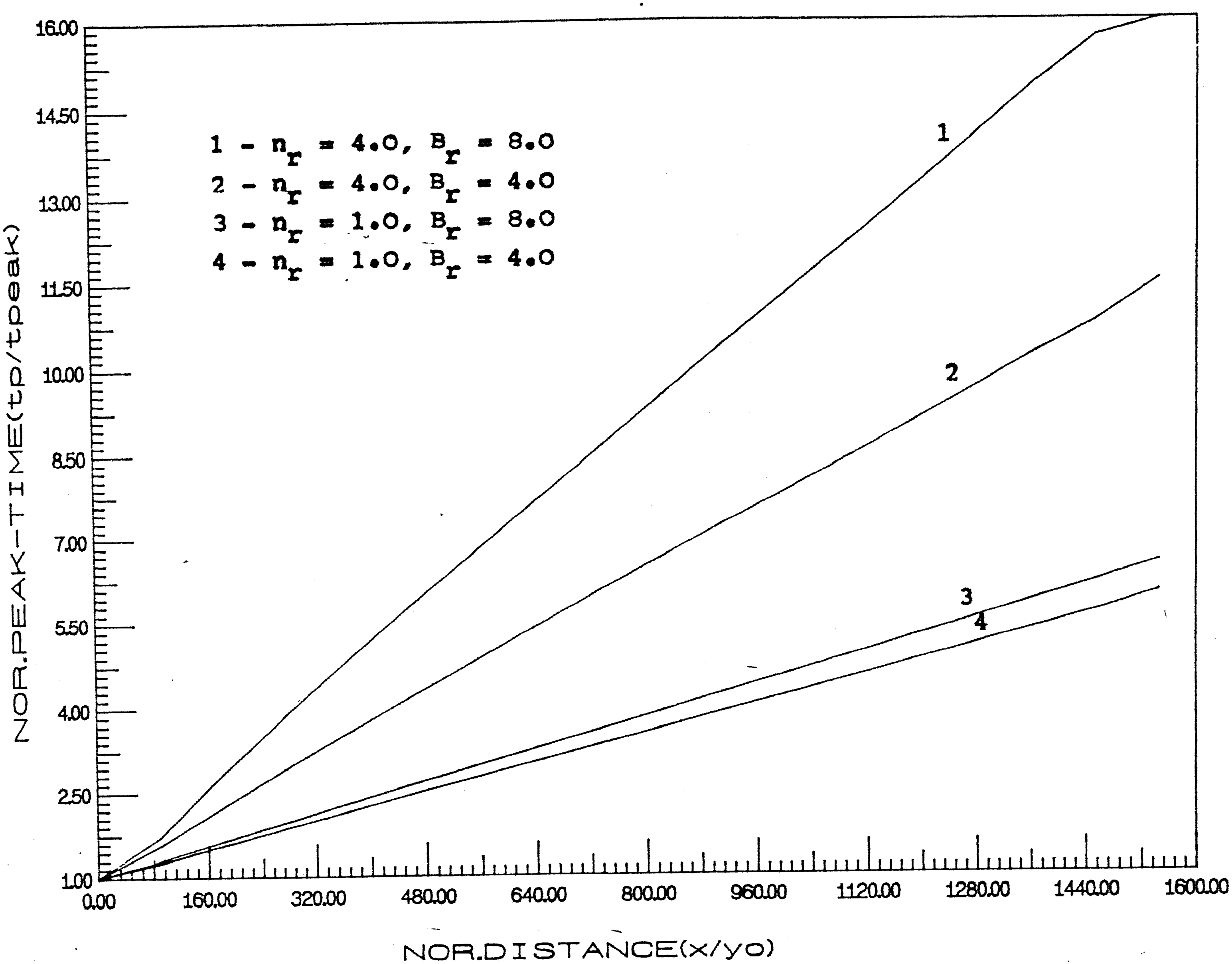


Fig. 3.9(b) Normalized time of peak depth vs normalized distance along the flood plain.

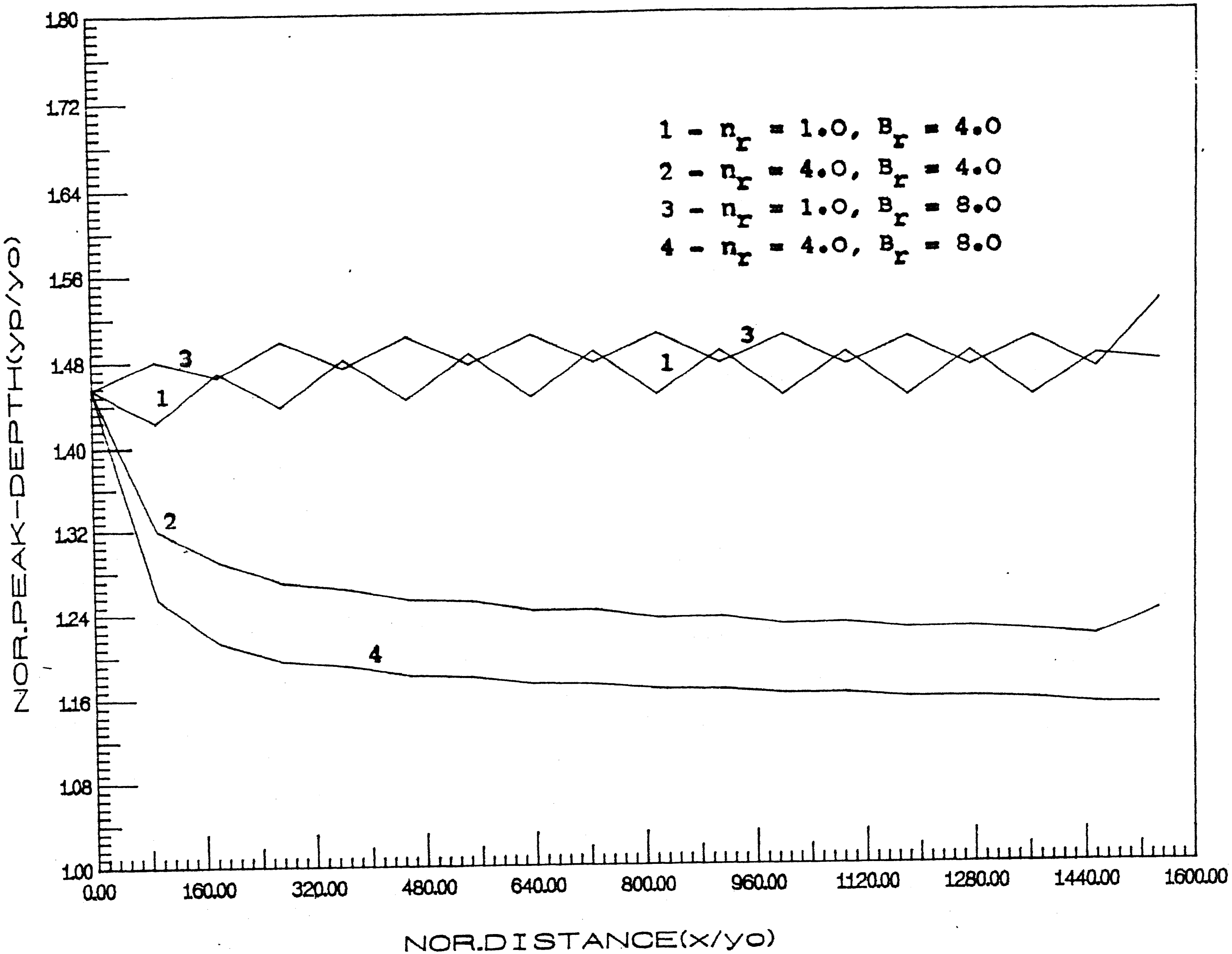


Fig. 3.10(a) **Normalized peak depth vs normalized distance along the main channel.**

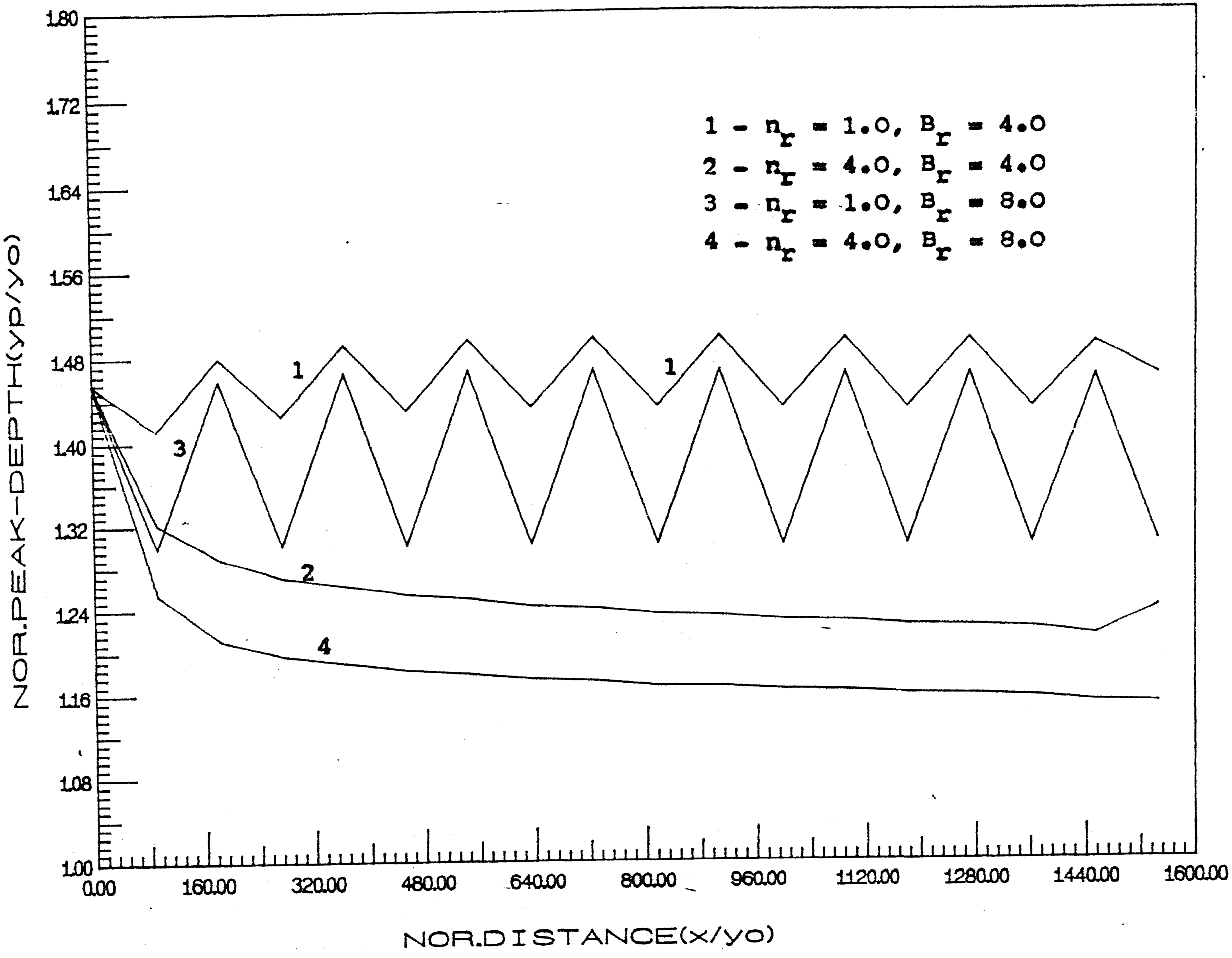


Fig. 3.10(b) Normalized peak depth vs normalized distance along the flood plain.

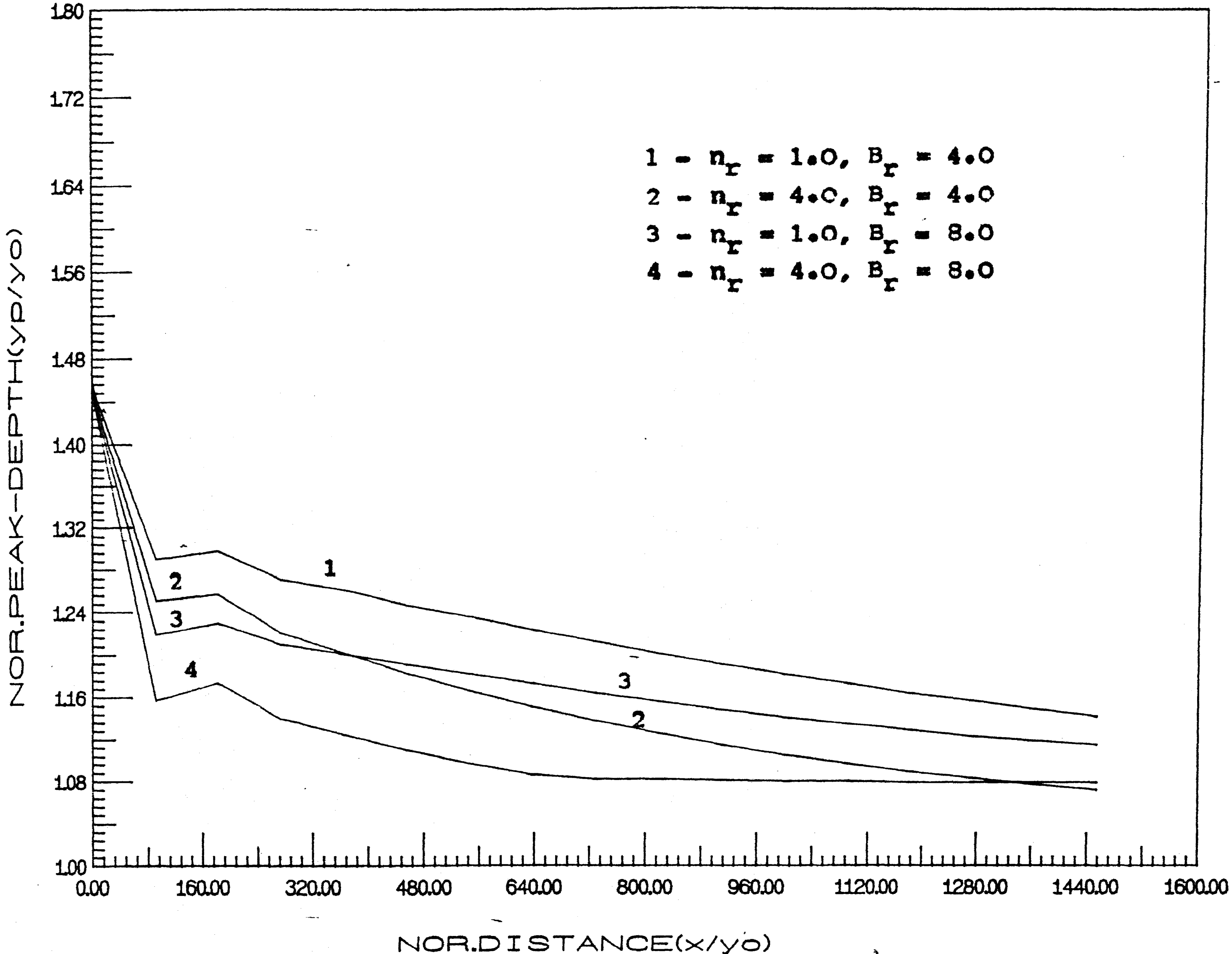


Fig. 3.10(c) Normalized peak depth vs normalized distance for one - dimensional model.

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APPENDIX A

```
C       THE PROGRAM FOR FLOOD ROUTINE THROUGH A RIVER WITH FLOOD PLAINS.
C       ****************************************************************
C       EXPLICIT METHOD HAS BEEN USED TO SOLVE THE DIFFERENCE EQUATION.
C       RUNGE-KUTTA OF FOURTH ORDER HAS BEEN USED TO SOLVE O.D.E.
C       THIS PROGRAM CAN BE RUN DIRECTLY FOR THE AREA HAVING AT LEAST
C       THREE FLOOD PLAIN CELLS EITER SIDE OF RIVER CELL.
C       FOR LESS NUMBER OF FLOOD PLAIN CELLS SLIGHT MODIFICATION IS
C       NEEDED WHILE DEFINIG THE IDENTIFIERS i.e,IDENT(J).
C       ----------------------------------------------------------------
C       NOTATIONS USED IN THE PROGRAM
C
C       A          = CROSS SECTIONAL AREA BETWEEN TWO CELLS.
C       CDO        = COEFFICIENT OF ORIFICE.
C       CDW        = COEFFICIENT OF WEIR.
C       CCR,CCF    = WEIGHTING FACTORS
C       DX         = LENGTH OF EACH CELL
C       DT         = TIME INTERVAL FOR COMPUTATIONS.
C       DL         = WIDTH OF EACH CELL
C       DELT       = TIME INTERVAL FOR WRITING THE RESULTS.
C       FRICH      = MANNING'S ROUGHNESS OF RIVER CELLS
C       FRICF      = MANNING'S ROUGHNESS OF FLOOD PLAIN CELLS.
C       G          = ACCELERATION DUE TO GRAVITY.
C       IN         = TOTAL NO  OF CELLS IN LONGITUDINAL DIRECTION.
C       ICI,ICP,   = THE NODES ALONG LENGTH WHERE STAGES ARE BEING CALCULATED.
C       JN         = TOTAL NO  OF CELLS IN TRANSVERSE DIRECTION.
C       JCAN       = ROW NO. OF RIVER CELL.
C       PM         = WETTED PERIMETER FOR RIVER CELLS.
C       PF         = WETTED PERIMETER FOR FLOOD PLAIN CELLS
C       QL         = FLOW ASSOCIATED WITH LEFT FACE OF CELL.
C       QR         = FLOW ASSOCIATED WITH RIGHT FACE OF CELL.
C       QT         = FLOW ASSOCIATED WITH TOP FACE OF CELL.
C       QB         = FLOW ASSOCIATED WITH BOTTEM FACE OF CELL.
C       SOM        = LONGITUDINAL BED SLOPE FOR RIVER.
C       SOF        = LONGITUDINAL BED SLOPE FOR FLOOD PLAIN.
C       T          = TIME OF COMPUTATION
C       TBASE      = BASE OF THE STAGE HYDROGRAPH.
C       TPEAK      = TIME UPTO PEAK OF THE HYDROGRAPH.
C       TLAST      = TIME UPTO COMPUTATION IS REQUIRED
C       TMAX       = TIME TO REACH PEAK STAGE IN THE CELL
C       YO         = INITIAL DEPTH IN RIVER CELLS.
C       YOF        = INITIAL DEPTH IN FLOOD PLAIN
C       YF         = HEIGHT OF RIVER BANK
C       Y          = DEPTH OVER CELLS AT KNOWN TIME STEP
C       YN         = DEPTH OVER CELLS AT NEXT TIME STEP
C       ZWO        = STEADY STATE STAGE OF HYDROGRAPH.
C       ZWOP       = PEAK STAGE OF HYDROGRAPH.
C       ZB         = R L OF BED
C       ZW         = ELEVATION OF WATER AT KNOWN TIME STEP.
C       ZWN        = ELEVATION OF WATER AT NEXT TIME STEP.
C       ZWMAX      = PEAK STAGE IN CELL
C       ----------------------------------------------------------------
C
        DIMENSION ZB(30,30),ZW(30,30),Y(30,30),ZWN(30,30)
        DIMENSION YN(30,30),IDENT(30),AK1(30,30),AK2(30,30)
        DIMENSION AK3(30,30),AK4(30,30),Y1(30,30),Y2(30,30)
        DIMENSION Y3(30,30),ZW1(30,30),ZW2(30,30),ZW3(30,30)
        DIMENSION ZWMAX(30,30),TMAX(30,30)
C       ----------------------------------------------------------------
        OPEN (UNIT=5,FILE='RK41.DAT')
        OPEN (UNIT=26,FILE='MURTY.OUT')
C       ----------------------------------------------------------------
        READ(5,*) IN,JN,DX,DL,SO,DT,DELT,JCAN
        READ(5,*) YO,YF,FRICH,FRICF,CCR,CCF
        READ(5,*) TPEAK,TBASE,TLAST,ZWO,ZWOP
```

```
      READ(5,*) IC1,IC2,IC3,IC4,IC5,IC6
      READ(5,*) (IDENT(J),J=1,JN)
C     ------------------------------------------------------------
      WRITE(*,8) IN,JN,DX,DL,SO,DT,DELT,JCAN,YO,YF,FRICM,FRICF,CCR,
     1 CCF,TPEAK,TBASE,TLAST,ZWO,ZWOP,IC1,IC2,IC3,IC4,IC5,IC6
8     FORMAT('IN =',5X,I3,/,'JN =',5X,I2,/,'DX =',5X,F6.2,/,
     1 'DL  =',5X,F6.2,/,'SO =',5X,F8.6,/,
     2 'DT =',5X,F6.2,/,'DELT =',4X,F6.2,/,'JCAN =',4X,I2,/,
     3 'YO =',5X,F6.2,/,'YF =',5X,F6.2,/,'FRICM =',1X,F6.4,/,
     4 'FRICF =',1X,F6.4,/,'CCR,CCF =',1X,2F6.2,/,'TPEAK =',1X,F7.0,/,
     5 'TBASE =',1X,F7.0,/,'TLAST =',1X,F7.0,/,'ZWO =',3X,F6.3,/,
     6 'ZWOP =',2X,F6.3,/,'IC1,IC2,IC3,IC4,IC5,IC6  =',2X,6I3)
      WRITE(*,9) (IDENT(J),J=1,JN)
9     FORMAT('IDENT(J) =',2X,9I2)
C     ------------------------------------------------------------
      PM=DL+2.0*YF
      PF=DL
      SOM=(1.0/FRICM)*SQRT(SO)
      SOF=(1.0/FRICF)*SQRT(SO)
      RM=(DL+YO)/PM
      YOF=YO-YF
      RF=(DL+YOF)/PF
      VO=SOM*RM**0.667
      VOF=SOF*RF**0.667
      T = 0.0
      TINT=0.0
C     ------------------------------------------------------------
C     INITIAL CONDITIONS
C     ------------------------------------------------------------
      DO 10 J=1,JN
      ZW(1,J)=ZWO
      IF(IDENT(J).NE.1) THEN
      Y(1,J)=YO-YF
      ZB(1,J)=ZW(1,J)-Y(1,J)
      ELSE
      Y(1,J)=YO
      ZB(1,J)=ZW(1,J)-Y(1,J)
      ENDIF
10    CONTINUE
      DO 15 I=2,IN
      DO 15 J=1,JN
      ZWMAX(I,J)=0.0
      ZW(I,J)=ZW(I-1,J)-DX*SO
      IF(IDENT(J).NE.1) THEN
      Y(I,J) = YO-YF
      ELSE
      Y(I,J)=YO
      ENDIF
      ZB(I,J)=ZW(I,J)-Y(I,J)
15    CONTINUE
C     ------------------------------------------------------------
C     UNSTEADY CONDITIONS
C
C     UPSTREAM BOUNDRY BY STAGE HYDROGRAPH
C     ------------------------------------------------------------
5     CONTINUE
      IF(T.LE.TPEAK) THEN
      TERM1=(ZWOP-ZWO)/TPEAK
      ZWOUN=ZWO+TERM1*T
      ELSE
      TERM1=(ZWOP-ZWO)/(TBASE-TPEAK)
      ZWOUN=ZWOP-TERM1*(T-TPEAK)
      ENDIF
      IF(T.GT.TBASE) ZWOUN=ZWO
      DO 20 J=1,JN
      ZWN(1,J)=ZWOUN
```

```
      IF(IDENT(I).NE.1) THEN
      YN(1,J)=((ZWDUN-ZWO)*(YO-YF))/YOF
      ELSE
      YN(1,J)=((ZWDUN-ZWO)+YO)/YO
      ENDIF
20    CONTINUE
C     -----------------------------------------------------------------
C     UNSTEADY CALCULATIONS AT INTERIOR NODES
C     -----------------------------------------------------------------
      DO 25 I=2,IN-1
      DO 25 J=JCAN,JN
      YY=Y(I,J)
      YL=Y(I-1,J)
      YR=Y(I+1,J)
      YT=Y(I,J+1)
      YB=Y(I,J-1)
      ZZW=ZW(I,J)
      ZWT=ZW(I,J+1)
      ZWB=ZW(I,J-1)
      ZZB=ZB(I,J)
      ZBT=ZB(I,J+1)
      K=IDENT(J)
      GO TO (30,35,40,45),K
30    CONTINUE
      CALL RIVCELL(DHDT,YY,YL,YR,YT,ZZW,ZWT,ZBT,DX,DL,PM,SOM,CCR,
     1 CCF)
      AK1(I,J)=DHDT
      GOTO 50
35    CONTINUE
      CALL NEXTRIV(DHDT,YY,YL,YR,YT,ZZW,ZWB,ZWT,ZZB,DX,DL,PF,SOF,FRICF,
     1 CCR,CCF)
      AK1(I,J)=DHDT
      GOTO 50
40    CONTINUE
      CALL INTERMED(DHDT,YY,YL,YR,YT,YB,ZZW,ZWT,ZWB,DX,DL,PF,SOF,FRICF,
     1 CCR,CCF)
      AK1(I,J)=DHDT
      GOTO 50
45    CONTINUE
      CALL SIDECELL(DHDT,YY,YL,YR,YB,ZZW,ZWB,DX,DL,PF,SOF,FRICF,CCR,
     1 CCF)
      AK1(I,J)=DHDT
50    CONTINUE
      Y1(I,J)=Y(I,J)+0.5*DT*AK1(I,J)
      ZW1(I,J)=ZW(I,J)+0.5*DT*AK1(I,J)
25    CONTINUE
      DO 125 I=2,IN-1
      DO 125 J=JCAN,JN
      Y1(1,J)=Y(1,J)
      Y1(IN,J)=Y(IN,J).
      YY=Y1(I,J)
      YL=Y1(I-1,J)
      YR=Y1(I+1,J)
      YT=Y1(I,J+1)
      YB=Y1(I,J-1)
      ZZW=ZW1(I,J)
      ZWT=ZW1(I,J+1)
      ZWB=ZW1(I,J-1)
      ZZB=ZB(I,J)
      ZBT=ZB(I,J+1)
      K=IDENT(J)
      GO TO (130,135,140,145),K
130   CONTINUE
      CALL RIVCELL(DHDT,YY,YL,YR,YT,ZZW,ZWT,ZBT,DX,DL,PM,SOM,CCR,
     1 CCF)
      AK2(I,J)=DHDT
```

```
      GOTO 150
135   CONTINUE
      CALL NEXTRIV(DHDT,YY,YL,YR,YT,ZZW,ZWB,ZWT,ZZB,DX,DL,PF,SOF,FRICF,
     1 CCR,CCF)
      AK2(I,J)=DHDT
      GOTO 150
140   CONTINUE
      CALL INTERMED(DHDT,YY,YL,YR,YT,YB,ZZW,ZWT,ZWB,DX,DL,PF,SOF,FRICF,
     1 CCR,CCF)
      AK2(I,J)=DHDT
      GOTO 150
145   CONTINUE
      CALL SIDECELL(DHDT,YY,YL,YR,YB,ZZW,ZWB,DX,DL,PF,SOF,FRICF,CCR,
     1 CCF)
      AK2(I,J)=DHDT
150   CONTINUE
      Y2(I,J)=Y(I,J)+0.5*DT*AK2(I,J)
      ZW2(I,J)=ZW(I,J)+0.5*DT*AK2(I,J)
125   CONTINUE
      DO 225 I=2,IN-1
      DO 225 J=JCAN,JN
      Y2(1,J)=Y(1,J)
      Y2(IN,J)=Y(IN,J)
      YY=Y2(I,J)
      YL=Y2(I-1,J)
      YR=Y2(I+1,J)
      YT=Y2(I,J+1)
      YB=Y2(I,J-1)
      ZZW=ZW2(I,J)
      ZWT=ZW2(I,J+1)
      ZWB=ZW2(I,J-1)
      ZZB=ZB(I,J)
      ZBT=ZB(I,J+1)
      K=IDENT(J)
      GO TO (230,235,240,245),K
230   CONTINUE
      CALL RIVCELL(DHDT,YY,YL,YR,YT,ZZW,ZWT,ZBT,DX,DL,PM,SOM,CCR,
     1 CCF)
      AK3(I,J)=DHDT
      GOTO 250
235   CONTINUE
      CALL NEXTRIV(DHDT,YY,YL,YR,YT,ZZW,ZWB,ZWT,ZZB,DX,DL,PF,SOF,FRICF,
     1 CCR,CCF)
      AK3(I,J)=DHDT
      GOTO 250
240   CONTINUE
      CALL INTERMED(DHDT,YY,YL,YR,YT,YB,ZZW,ZWT,ZWB,DX,DL,PF,SOF,FRICF,
     1 CCR,CCF)
      AK3(I,J)=DHDT
      GOTO 250
245   CONTINUE
      CALL SIDECELL(DHDT,YY,YL,YR,YB,ZZW,ZWB,DX,DL,PF,SOF,FRICF,CCR,
     1 CCF)
      AK3(I,J)=DHDT
      GOTO 250
250   CONTINUE
      Y3(I,J)=Y(I,J)+DT*AK3(I,J)
      ZW3(I,J)=ZW(I,J)+DT*AK3(I,J)
225   CONTINUE
      DO 325 I=2,IN-1
      DO 325 J=JCAN,JN
      Y3(1,J)=Y(1,J)
      Y3(IN,J)=Y(IN,J)
      YY=Y3(I,J)
      YL=Y3(I-1,J)
      YR=Y3(I+1,J)
```

```
      YT=Y3(I,J+1)
      YB=Y3(I,J-1)
      ZZW=ZW3(I,J)
      ZWT=ZW3(I,J+1)
      ZWB=ZW3(I,J-1)
      ZZB=ZB(I,J)
      ZBT=ZB(I,J+1)
      K=IDENT(J)
      GO TO (330,335,340,345),K
330   CONTINUE
      CALL RIVCELL(DHDT,YY,YL,YR,YT,ZZW,ZWT,ZBT,DX,DL,PM,SOM,CCR,
     1 CCF)
      AK4(I,J)=DHDT
      GOTO 350
335   CONTINUE
      CALL NEXTRIV(DHDT,YY,YL,YR,YT,ZZW,ZWB,ZWT,ZZB,DX,DL,PF,SOF,FRICF,
     1 CCR,CCF)
      AK4(I,J)=DHDT
      GOTO 350
340   CONTINUE
      CALL INTERMED(DHDT,YY,YL,YR,YT,YB,ZZW,ZWT,ZWB,DX,DL,PF,SOF,FRICF,
     1 CCR,CCF)
      AK4(I,J)=DHDT
      GOTO 350
345   CONTINUE
      CALL SIDECELL(DHDT,YY,YL,YR,YB,ZZW,ZWB,DX,DL,PF,SOF,FRICF,CCR,
     1 CCF)
      AK4(I,J)=DHDT
      GOTO 350
350   CONTINUE
325   CONTINUE
      DO 400 I=2,IN-1
      DO 400 J=JCAN,JN
      DZW=1/6.0*(AK1(I,J)+2.0*AK2(I,J)+2.0*AK3(I,J)+AK4(I,J))
      ZWN(I,J)=ZW(I,J)+DZW
C     DEPTH NORMALISED
      IF (IDENT(J).NE.1) THEN
      YN(I,J)=(ZWN(I,J)-ZB(I,J))/YOF
      ELSE
      YN(I,J)=(ZWN(I,J)-ZB(I,J))/YO
      ENDIF
400   CONTINUE
C     -----------------------------------------------------------------
C     DOWNSTREAM BOUNDRY CONDITION
C     -----------------------------------------------------------------
      DO 55 J=JCAN,JN
      ZWN(IN,J)=ZWN(IN-1,J)-SO*DX
      IF (IDENT(J).NE.1) THEN
      YN(IN,J)=(ZWN(IN,J)-ZB(IN,J))/YOF
      ELSE
      YN(IN,J)=(ZWN(IN,J)-ZB(IN,J))/YO
      ENDIF
55    CONTINUE
C     -----------------------------------------------------------------
C     RESETTING THE VALUES
C     -----------------------------------------------------------------
      DO 60 I=1,IN
      DO 60 J=JCAN,JN
      ZW(I,J)=ZWN(I,J)
      IF (IDENT(J).NE.1) THEN
      Y(I,J)=YN(I,J)*YOF
      ELSE
      Y(I,J)=YN(I,J)*YO
      ENDIF
60    CONTINUE
C     NORMALISED PARAMETERS
```

```
      TN=T/TPEAK
C     ----------------------------------------------------------------
C     WRITING THE RESULTS
C     ----------------------------------------------------------------
      IF(T EQ TINT) THEN
      WRITE(*,65)TN,YN(IC2,JCAN),YN(IC3,JCAN),
     1 YN(IC4,JCAN),YN(IC5,JCAN)
65    FORMAT(1X,5F8.4)
      WRITE(*,68)TN,YN(IC2,JCAN+2),YN(IC3,JCAN+2),
     1 YN(IC4,JCAN+2),YN(IC5,JCAN+2)
68    FORMAT(1X,5F8.4)
      TINT=TINT+DELT
      ELSE
      ENDIF
      DO 500 I=1,IN-1
      DO 500 J=1,JN
      IF(ZWN(I,J) GE.ZWMAX(I,J)) THEN
      ZWMAX(I,J)=ZWN(I,J)
      TMAX(I,J)=T
      ELSE
      ENDIF
500   CONTINUE
      T=T+DT
      IF(T LE TLAST) THEN
      GOTO 5
      ELSE
      WRITE(*,*)
      WRITE(*,*) ,'PEAK VALUES AND OCCURANCE TIME'
      WRITE(*,*)
      WRITE(*,*) 'AT IC=IC2 RIVER NEXTRIVER SIDE '
      WRITE(*,*) ZWMAX(IC2,JCAN),TMAX(IC2,JCAN)
      WRITE(*,*) ZWMAX(IC2,JCAN+1),TMAX(IC2,JCAN+1)
      WRITE(*,*) ZWMAX(IC2,JCAN+2),TMAX(IC2,JCAN+2)
      WRITE(*,*) ZWMAX(IC2,JCAN+3),TMAX(IC2,JCAN+3)
      WRITE(*,*) ZWMAX(IC2,JN),TMAX(IC2,JN)
      WRITE(*,*) '--------------------------------------------'
      WRITE(*,*) 'AT IC=IC3 RIVER NEXTRIVER SIDE '
      WRITE(*,*) ZWMAX(IC3,JCAN),TMAX(IC3,JCAN)
      WRITE(*,*) ZWMAX(IC3,JCAN+1),TMAX(IC3,JCAN+1)
      WRITE(*,*) ZWMAX(IC3,JCAN+2),TMAX(IC3,JCAN+2)
      WRITE(*,*) ZWMAX(IC3,JCAN+3),TMAX(IC3,JCAN+3)
      WRITE(*,*) ZWMAX(IC3,JN),TMAX(IC3,JN)
      WRITE(*,*)'--------------------------------------------'
      WRITE(*,*) 'AT IC=IC4 RIVER NEXTRIVER SIDE '
      WRITE(*,*) ZWMAX(IC4,JCAN),TMAX(IC4,JCAN)
      WRITE(*,*) ZWMAX(IC4,JCAN+1),TMAX(IC4,JCAN+1)
      WRITE(*,*) ZWMAX(IC4,JCAN+2),TMAX(IC4,JCAN+2)
      WRITE(*,*) ZWMAX(IC4,JCAN+3),TMAX(IC4,JCAN+3)
      WRITE(*,*) ZWMAX(IC4,JN),TMAX(IC4,JN)
      WRITE(*,*)'--------------------------------------------'
      WRITE(*,*) 'AT IC=IC5 RIVER NEXTRIVER SIDE '
      WRITE(*,*) ZWMAX(IC5,JCAN),TMAX(IC5,JCAN)
      WRITE(*,*) ZWMAX(IC5,JCAN+1),TMAX(IC5,JCAN+1)
      WRITE(*,*) ZWMAX(IC5,JCAN+2),TMAX(IC5,JCAN+2)
      WRITE(*,*) ZWMAX(IC5,JCAN+3),TMAX(IC5,JCAN+3)
      WRITE(*,*) ZWMAX(IC5,JN),TMAX(IC5,JN)
      WRITE(*,*)'--------------------------------------------'
      ENDIF
      STOP
      END
C     ----------------------------------------------------------------
C     SUBROUTINE STARTS FROM HERE
C     ----------------------------------------------------------------
C     RIVER CELL
C     ----------------------------------------------------------------
      SUBROUTINE RIVCELL(DHDT,YY,YL,YR,YT,ZZW,ZWT,ZBT,DX,DL,PM,SOM,CCR,
```

```
     1    CCF)
          C1=2.0/3.0*SQRT(2.0*9.81)*DX
          C2=1.5*C1
          C3=DX*DL
          A=DL*0.5*(YL+YY)
          R=A/PM
          QL=A*R**0.667*SOM
          A=DL*0.5*(YY+YR)
          R=A/PM
          QR=A*R**0.667*SOM
          IF(ZZW.GT.ZWT) THEN
          H=ZZW-ZWT
          W=YT
          CDW=0.611+0.075*H/W
          CDO=0.611-0.175*W/(H+W)
          IF((ZZW-ZBT).GT.1.5*YT) THEN
          CDO=0.0
          ELSE
          CDW=0.0
          ENDIF
          QT=CCR*(C1*CDW*H**1.5+C2*CDO*W*H**0.5)
          ELSE
          H=ZWT-ZZW
          W=ZZW-ZBT
          CDW=0.611+0.075*H/W
          CDO=0.611-0.175*W/(H+W)
          IF((ZZW-ZBT).LT.0.666*YT) THEN
          CDO=0.0
          ELSE
          CDW=0.0
          ENDIF
          QT=-CCR*(C1*CDW*H**1.5+C2*CDO*W*H**0.5)
          ENDIF
          DHDT=(QL-QR-2.0*QT)/C3
          RETURN
          END
C         ----------------------------------------------------------------
C         CELL NEXT TO THE RIVER
C         ----------------------------------------------------------------
          SUBROUTINE NEXTRIV(DHDT,YY,YL,YR,YT,ZZW,ZWB,ZWT,ZZB,DX,
          DL,PF,SOF,FRICF,CCR,CCF)
          C1=2.0/3.0*SQRT(2.0*9.81)*DX
          C2=1.5*C1
          C3=DX*DL
          A=DL*0.5*(YL+YY)
          R=A/PF
          QL=A*R**0.667*SOF
          A=DL*0.5*(YY+YR)
          R=A/PF
          QR=A*R**0.667*SOF
          IF(ZZW.LT.ZWB) THEN
          H=ZWB-ZZW
          W=YY
          CDW=0.611+0.075*H/W
          CDO=0.611-0.175*W/(H+W)
          IF(YY.LT.0.666*(ZWB-ZZB))THEN
          CDO=0.0
          ELSE
          CDW=0.0
          ENDIF
          QR=-CCR*(C1*CDW*H**1.5+C2*CDO*W*H**0.5)
          ELSE
          H=ZZW-ZWB
          W=ZWB-ZZB
          CDW=0.611+0.075*H/W
          CDO=0.611-0.175*W/(H+W)
```

```
      IF((ZWB-ZZB).LT.0.666*YY)THEN
      CDO=0.0
      ELSE
      CDW=0.0
      ENDIF
      QR=-CCR*(C1*CDW*H**1.5+C2*CDO*W*H**0.5)
      ENDIF
      A=DX*0.5*(YY+YT)
      R=A/DX
      IF(ZWT.LT.ZZW) THEN
      SLOPE=SQRT((ZZW-ZWT)/DL)/FRICF
      ELSE
      SLOPE=-SQRT((ZWT-ZZW)/DL)/FRICF
      ENDIF
      QT=CCF*A*R**0.667*SLOPE
      DHDT=(QL-QR+QB-QT)/C3
      RETURN
      END
C     ------------------------------------------------------------------
C     INTERMEDIATE FLOOD BANK CELL
C     ------------------------------------------------------------------
      SUBROUTINE INTERMED(DHDT,YY,YL,YR,YT,YB,ZZW,ZWT,ZWB,
     1 DX,DL,PF,SOF,FRICF,CCR,CCF)
      C1=2.0/3.0*SQRT(2.0*9.81)*DX
      C2=1.5*C1
      C3=DX*DL
      A=DL*0.5*(YL+YY)
      R=A/PF
      QL=A*R**0.667*SOF
      A=DL*0.5*(YY+YR)
      R=A/PF
      QR=A*R**0.667*SOF
      A=DX*0.5*(YY+YB)
      R=A/DX
      IF(ZZW.LT.ZWB) THEN
      SLOPE=SQRT((ZWB-ZZW)/DL)/FRICF
      ELSE
      SLOPE=-SQRT((ZZW-ZWB)/DL)/FRICF
      ENDIF
      QB=CCF*A*R**0.667*SLOPE
      A=DX*0.5*(YY+YT)
      R=A/DX
      IF(ZZW.GT.ZWT) THEN
      SLOPE=SQRT((ZZW-ZWT)/DL)/FRICF
      ELSE
      SLOPE=-SQRT((ZWT-ZZW)/DL)/FRICF
      ENDIF
      QT=CCF*A*R**0.667*SLOPE
      DHDT=(QL-QR+QB-QT)/C3
      RETURN
      END
C     ------------------------------------------------------------------
C     SIDE BOUNDARY CELL
C     ------------------------------------------------------------------
      SUBROUTINE SIDECELL(DHDT,YY,YL,YR,YB,ZZW,ZWB,DX,DL,PF,SOF,FRICF
     1 CCR,CCF)
      C1=2.0/3.0*SQRT(2.0*9.81)*DX
      C2=1.5*C1
      C3=DX*DL
      A=DL*0.5*(YL+YY)
      R=A/(PF+0.5*(YL+YY))
      QL=A*R**0.667*SOF
      A=DL*0.5*(YY+YR)
      R=A/(PF+0.5*(YY+YR))
      QR=A*R**0.667*SOF
      A=DX*0.5*(YY+YB)
```

```
[illegible]
IF(ZZW.LT.ZWB) THEN
SLOPE=SQRT((ZWB-ZZW)/DL)/FRICF
ELSE
SLOPE=-SQRT((ZZW-ZWB)/DL)/FRICF
ENDIF
QB=CIF*A*R**0.667*SLOPE
DHDT=(QL-QR+QB)/C3
RETURN
END
```